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The Building Collapse at
2000 Commonwealth Avenue
Boston, Massachusetts
on
January 25, 1971
Report of
The Mayor's Investigating Commission
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THE BUILDING COLLAPSE AT
2000 COMMONWEALTH AVENUE
BOSTON, MASSACHUSETTS

on

January 25, 1971

REPORT

OF

THE MAYOR'S INVESTIGATING COMMISSION

PART I

JUNE 1971

THE BUILDING COLLAPSE AT
2000 COMMONWEALTH AVENUE
BOSTON, MASSACHUSETTS

on

January 25, 1971

REPORT
OF
THE MAYOR'S INVESTIGATING COMMISSION

Mr. Raymond O. Granger
Mr. John W. Peirce
Mr. Herman G. Protze
Mr. John J. Tobin
Rev. Msgr. Francis J. Lally, Chairman

Consultants:
Nichols, Norton and Zaldastani, Inc.
The Thompson & Lichtner Co., Inc.
Counsel:
Mr. Sheldon M. Drucker

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by

The City of Boston, Massachusetts

INTRODUCTION

A commission, selected by Mayor Kevin White, met for the first time on February 1, 1971, just one week after the tragic collapse of a building which was under construction at 2000 Commonwealth Avenue, Boston, Massachusetts. The Commission was that day charged by the Mayor with an investigation into the cause of the collapse, and further requested to make recommendations, with particular reference to the City of Boston, that would guard against its repetition.

Members of the Commission were Mr. Raymond O. Granger, representing the Associated General Contractors of Massachusetts; Mr. John W. Peirce, representing the Boston Society of Architects; Mr. Herman G. Protze, representing the Boston Society of Civil Engineers; Mr. John J. Tobin, representing the Boston Building and Construction Trades Council; and Rev. Msgr. Francis J. Lally, Chairman.

The Mayor put at the disposal of the Commission Mr. Sheldon M. Drucker, Assistant Corporation Counsel for the City, who acted as counsel and secretary. Nichols, Norton and Zaldastani, Inc. of Boston was retained by the City to conduct a technical investigation of all aspects of the failure, to determine the cause of the collapse, and to make a report to the Commission which would include their technical findings as well as conclusions and recommendations. The Thompson & Lichtner Company, Inc. of Brookline

was retained by the City to sample and test concrete and steel reinforcement, to submit test results, and to state whether or not the concrete was a factor in the collapse.

Dr. Othar Zaldastani, of Nichols, Norton and Zaldastani, Inc. directed the technical effort. Dr. Miles N. Clair, of the Thompson & Lichtner Company, Inc. was responsible for laboratory and field testing of materials.

Apart from the detailed investigations, the Commission and its consultants met regularly at the office of the City Law Department for a total of 15 meetings. Mr. James Cosgrove, Supervisor of Building Plans in the Department of Public Safety of the Commonwealth, was an observer at most of these meetings.

At its first meeting the Commission inquired into the condition of the structure that remains standing at 2000 Commonwealth Avenue. The City demanded that the owner take measures to eliminate the immediate danger of further collapse. The owners' engineer evolved a plan to fence the site and shore the remaining structure with an adequate number and arrangement of temporary shores. These temporary measures have been completed and a decision on the future of the structure is pending. There can be no further action without approval of the Building Commissioner.

At following meetings the Commission met with 24 persons who were invited to offer testimony on various aspects of the case, all of whom appeared voluntarily, some with counsel. Written and oral statements of several other individuals were made available. From those present at the time of the building failure, a pattern of testimony developed which was virtually identical in all essential aspects, and an extensive array of further testimony was considered unnecessary.

It was the judgment of the Commission that the interests of the City would be most effectively served by presenting a single report, including both technical and non-technical aspects. Since a major part of the investigation was technical in nature, the Commission requested Dr. Zaldastani to incorporate all aspects of the investigation into a draft report. Dr. William A. Litle, Associate Professor of Civil Engineering at the Massachusetts Institute of Technology, who was retained by Dr. Zaldastani to assist in the technical investigation, prepared this draft. The Commission reviewed the draft in detail and all members, with the consultants, participated in the preparation of the final report.

The report is in two parts. Part I contains a description of the developments of the project at 2000 Commonwealth Avenue, a description of the failure; the cause of the collapse; a catalog of irregularities and deficiencies in the development, design, construction, and inspection of the project; and finally, conclusions and recommendations. The essential technical results are incorporated in Part I. Part II presents, in more depth and detail, all aspects of the technical investigation and analysis.

All members of the Commission are fully satisfied that the report presented to the Mayor represents a product of the best evidence, information, and advice available to them within the scope of the investigation as set by the Mayor.

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Explanation

Please Read

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COMPUTER ANALYSES

AVAILABLE

from

DR. WILLIAM LITTLE

MASSACHUSETTS INSTITUTE OF TECHNOLOGY

BUILDING # 1 - 142

Explanation

Please Read

APPENDIX II. 5
TRANSCRIPTS OF
TESTIMONY FROM
24 WITNESSES

NOT AVAILABLE

1.

THE PROJECT DEVELOPMENT

A. INITIAL BUILDING ATTEMPTS

Building Department records for the property at 2000 Commonwealth Avenue, Boston, Massachusetts indicate that the first attempt to build an apartment house there was made in 1964. A building permit application was filed on November 3, 1964, for a seven story, eighty-nine apartment dwelling. The owner of the property at that time was Bartholomew W. Cosentino, 1580 Commonwealth Avenue, West Newton, Massachusetts. His architect for the project was S. S. Eisenberg of 739 Boylston Street, Boston, and his licensed builder was William J. Lamborghini of La Mont Corporation, 1105 Washington Street, West Newton, Massachusetts. The building permit application was made by Herbert W. Eisenberg of the above architectural firm.

This permit application was refused because the allowable building height exceeded that specified in Section 13 of Chapter 488 of the Acts of 1924, the then effective zoning law. Appeal of the refusal was made to the Board of Appeal which granted a zoning variance on November 18, 1964. Thereupon, the permit was issued on December 24, 1964.

The next entry in the Building Department file is a letter dated May 24, 1965, from W. J. Lamborghini of La Mont Corporation stating that construction of

the building was started on May 22, 1965. Subsequent to this, records in the Building Department show that an excavation was made and the lot fenced. More than two years later, on August 16, 1967, notice was given to Mr. Cosentino by the Building Department that his permit had lapsed. This action was taken pursuant to Section 111 (e) of the Boston Building Code which permitted the Building Commissioner to revoke a permit if after commencement of the work there is unreasonable delay in completing the building.

Bartholomew W. Cosentino filed a new building permit application on November 20, 1967, for a fourteen story, eighty-five apartment, reinforced concrete and brick luxury apartment house to be known as the St. Moritz Apartments. This application named George Garfinkle Associates of 137 Newbury Street, Boston, as architect.

Section 110 (b) of the Boston Building Code requires that "before a permit is issued such application shall also be signed by the person who is to perform or take charge of the work covered by such permit." Since this application was not signed by such a person, the permit was not issued. Pursuant to Section 110 (g) the application later was deemed abandoned since no permit was issued within six months of the date of application.

B. DEVELOPMENT OF THE CURRENT PROJECT

Apparently Mr. Cosentino did not complete the building permit application for the St. Moritz Apartments because the costs were too high to enable the project to be economically viable.

Although it did not become a matter of record until December 1968, it appears that Mr. Cosentino arranged in late 1967 or early 1968 to bring several additional partners into the ownership of 2000 Commonwealth Avenue.

A Saugus, Massachusetts architect, Phillip J. Burne, was consulted regarding the question of how the property could best be developed. Mr. Burne suggested that the site would need to be more densely developed and prepared for the owners some schematic density calculations. A zoning change for the property, from zone H2 to H3, was obtained on July 3, 1968. The presentation of this zoning case was by Mr. Harold Katz, who became an owner of record in December 1968.

It has been stated to the Commission that the new owners used a letter form of agreement to engage the architectural firm of Webb, Zerafa and Menkes, 2050 Rue Mansfield, Montreal, Canada, to design yet another apartment building. Webb, Zerafa and Menkes presumably engaged M. S. Yolles and Associates, 1420 Sherbrooke Street, West Toronto, Canada as structural engineers and Brais, Frigon, Hanley, Brett & Rinfret, Consulting Engineers of Montreal, Canada as mechanical and electrical engineers. These firms designed a sixteen story reinforced concrete apartment dwelling with a two-level underground parking garage.

On December 23, 1968, Bartholomew W. Cosentino conveyed the property at 2000 Commonwealth Avenue to Wilfred Fink, Toronto, Canada; Morris Feinstein and Theodore Hurwitz, both of Montreal, Canada; and Arthur Haffer, Bartholomew W. Cosentino, Harold Katz, Stanley H. Rudman, and Lewis G. Pollock, all of Massachusetts, as general partners. They had made a partnership agreement on October 18, 1968.

One set of plans for the sixteen story building carries an architect's note "issued for permit 7-3-69". On August 1, 1969, the Building Department issued an excavation permit to B. W. Cosentino as authorized agent for the owner and Toulon Construction Co. as contractor.

A building permit application, dated August 27, 1969, and signed by Jack Palevsky as Owner or Authorized Agent for an Owner named as 2000 Commonwealth Associates, was received by the Building Department on August 28, 1969. The application stated an estimated construction cost of $3,700,000. The contractor named on the application was Toulon Construction, Inc., 4018 St. Catherine W., Montreal, Quebec. The signature of Dante F. Montouri of Cochituate, Massachusetts, a registered engineer in Massachusetts, was on the application in the space provided for the signature of a licensed builder.

The structural plans were drawn by M. S. Yolles and Associates. A sworn affidavit, dated August 29, 1969, and stating that the plans and computations for the sixteen story building were in accordance with the Building Code of the City of Boston, was filed with the building permit application. This affidavit was signed by P. J. Quinn, professional engineer, as Vice President of the Yolles firm. The affidavit was notarized by Victor G. Dragone of Everett, Massachusetts.

September 4, 1969, marked yet another change in ownership. Two of the previous owners dropped out and three new ones were added. Under agreement of that date, the aforementioned owners, as general partners, conveyed the property at 2000 Commonwealth Avenue to Wilfred Fink; Morris Feinstein; Theodore Hurwitz; Arthur Haffer; Bartholomew Cosentino; Harold Katz; and John Alper, Jack Palevsky, and Bernard (Baril) Palevsky, all of Montreal, Canada, as general partners. A deed recorded this transaction on October 22, 1969. Messrs. John Alper, Jack Palevsky and Bernard Palevsky are also the three principals of Toulon Construction, Inc.

The building permit was granted on September 5, 1969. There are several irregularities and discrepancies relating to the events leading to the

issuance of this permit. Those that the Commission has uncovered are enumerated in Section 4A.

During the course of construction, there was an additional change of concept and of ownership. The Owners decided to sell the apartments as condominiums rather than retaining and renting them. The same Phillip J. Burne of Saugus, Massachusetts, who is mentioned earlier, was retained to help in this sales effort. Some modifications of layout were made and steps were being taken to sell the condominiums. A sales brochure was prepared but no units had been sold.

On November 10, 1970, ownership again changed by means of a conveyance from the nine general partners to Milton Adess, Bartholomew Cosentino, and Larry L. Webb, all of Massachusetts, as trustees of the 2000 Commonwealth Association Trust. Listed as beneficiary of the Trust is the 2000 Commonwealth Association, a joint venture. The Commission requested Mr. James DeGiacomo, attorney for Mr. Cosentino, to furnish the names of the individual beneficiaries, and he refused to do so.

On November 10, 1970, a mortgage deed recorded that the trustees obtained a construction loan of $3,000,000. from the Northeast Federal Savings and Loan Association, Watertown, Massachusetts. On January 4, 1971, a second mortgage deed recorded that a loan in the amount of $2,000,000. was obtained from Bartholomew W. and Anna Cosentino as trustees of C. F. Realty Trust. These mortgages are still outstanding.

C. GENERAL DESCRIPTION OF THE SIXTEEN STORY BUILDING

The structure of the apartment building at 2000 Commonwealth Avenue was a cast-in-place reinforced concrete flat slab construction with a central elevator core.

The design called for all columns to be founded on spread footings on hardpan capable of supporting 10 tons per square foot or sound rock and for the elevator core walls to be founded on strip footings resting on sound rock. This foundation design was based upon fourteen soil borings which indicated the presence of such soil conditions.

The structure incorporated two levels of underground parking. The first (ground) floor included a swimming pool, other ancillary spaces, and one apartment. The second through sixteenth floors were to include 132 apartments. Mechanical equipment was to be located on a mechanical room (penthouse) floor 5 feet 2 inches above the main roof. The top of the building was to be a roof over the mechanical room.

Figure 1 shows an architect's rendering of the building which was included in the condominium sales brochure and the photograph in Figure 2 shows the extent of construction two hours prior to the collapse. Figure 3 shows the overall plan dimensions of the structure. Figure 4 shows the location of columns and balconies for the typical (3rd through 16th) floors and Figure 5 shows the main roof and mechanical room floor.

From the 11th floor to the main roof, columns C5, C8, H5, and H8 had cross-sections 12 x 84 inches; columns E2, E11, F2, and F11 had cross-sections

12 x 29 inches; and all other columns had cross-sections 12 x 24 inches. From the 11th floor to the sub-basement the column cross-sections increased by varying amounts at different levels. Exterior columns were faced with brick.

The elevator core had inside dimensions of 15 feet 8 inches by 22 feet 0 inches and its reinforced concrete walls were 10 inches thick.

The flat slabs for the levels from the 2nd floor to the main roof were generally designed to be 7 1/2 inches thick. However, in the bays directly north and south of the elevator core and the bays where the two stairwells are located, the slab design thickness was 9 inches. The design thickness of balcony slabs for the 2nd to 16th floor levels varied from 4 1/2 to 6 inches. Concrete block walls were to surround the stairwells. The mechanical room floor slab was designed to be 9 inches thick.

From the 2nd to the 16th floor the story height was 9 feet. From the 16th floor to the main roof the story height was 9 feet 6 inches. The mechanical room floor was 5 feet 2 inches above the main roof and its roof was intended to be yet another 12 feet higher. The total design height from the roof of the mechanical room to the ground floor was 169 feet 5 inches.

D. ARRANGEMENTS FOR CONSTRUCTION

The General Contractor was Toulon Construction, Inc. , of Montreal, Canada. Toulon installed Mr. Pierre Fontaine as job superintendent and he was their only employee at the site. Virtually all work was subcontracted and all subcontracts which have come to the attention of the Commission were between

the Owner and Subcontractor and did not directly involve Toulon. When Mr. Fontaine needed men to meet Toulon's responsibilities, he borrowed them from the subcontractors and was backcharged.

The Franki Foundation Company of Boston was retained to underpin the foundations of the existing building on the east side of 2000 Commonwealth Avenue during construction of the deep basement.

The structural concrete work was subcontracted to Fountain Construction Company, a New York corporation, with Mr. Boris Kanieff assigned as the general superintendent. The terms of the contract between the Owner and Fountain did not require Fountain to provide any inspection or any of the cold weather protection measures specified by the designers for concrete curing. From time to time Toulon retained Hub Testing Laboratories, Waltham, Massachusetts, to inspect the concrete. J. H. McNamara, Inc., Boston, Massachusetts, supplied the ready-mix concrete and provided the basic concrete mix designs. Northern Steel, Inc., Boston, supplied the reinforcing steel.

Other known subcontracts were to Pizzotti Brothers, Inc., Saugus, Massachusetts, for masonry work, Ashley Electric Corporation, Boston, for electrical work; Loustan Contracting Corporation, Boston, for plumbing, heating, ventilation, and air-conditioning; Gate-City Dry-Wall Co., Inc., Nashua, New Hampshire, for interior partitions; Dorel Steel Corporation, North Quincy, Massachusetts, for ornamental ironwork; and Rusco Industries, for steel windows and sliding doors.

E. CONSTRUCTION

The partial excavation from the 1965 construction start was enlarged late in the fall of 1969 and, with the exception of a concrete truck drivers' strike in May, June, and July 1970, work proceeded continuously until January 25,1971. Apparently there were no major difficulties arising as a consequence of the construction until the day of the collapse.

On January 25,1971, concrete was being placed in the mechanical room floor slab and in the walls, wall beams, and brackets which extend from the roof slab up to the mechanical floor slab. A crane, stationed on the south side of the building, was used to lift the concrete to the mechanical room floor level. The extent of this floor slab is shown on Figure 5.

Brick work had progressed to the 16th floor level and the building was essentially enclosed from the 2nd to the 15th floors. Parapets for the balcony slabs were completed up through the 10th floor level. Rough plumbing and lines for the heating, ventilating, and air-conditioning system were in varying stages of completion all the way up to the main roof level. Interior apartment partitions were being installed on the lower floors and some bathtubs had been placed. Electricians were running lines, installing panels, etc., up to and on the 16th floor. A temporary construction elevator was located at the south edge of the building between lines 7 and 8. This elevator was used to hoist materials for the subcontractors.

On the basis of records furnished the Commission by some of the subcontractors, it is estimated that about 100 men were working in or around the building at the time of the failure. At the 16th floor, main roof, and mechanical room floor levels workmen were concentrated between lines 2 and 7. This group,

numbering perhaps 30, consisted of bricklayers, carpenters, ironworkers, cement finishers, electricians, laborers, and a steamfitter. The Commission knows that a few men were on the ground and presumes that the other workmen were distributed throughout the building.

2.

A DESCRIPTION OF THE FAILURE

The failure of the structure at 2000 Commonwealth Avenue proceeded through three distinct phases. First, eyewitnesses noted that the roof slab failed in shear around interior column E5 and that the deflection of the roof slab in span E5-E6 progressively increased until a sag of 4 feet occurred.

In the second phase the initial failure suddenly propagated laterally to include the entire eastern half of the roof slab. Essentially all of the roof from lines 3 to 6 collapsed onto the 16th floor. The roof slab did not sever through the columns along line 2 or at some of the columns along line D. The photographs in Figure 6, taken from ground level off the northeast corner of the structure, show the collapse at this state. Tipping of the roof slab over the line 2 columns is clearly seen. Columns G3 and G4 are seen sticking up in Figures 6b and 6c with no remnant of the roof slab attached to their tops.

After a period variously estimated at between 10 and 20 minutes, the third and final phase commenced with a failure in the 16th floor slab. This slab collapsed onto the 15th floor, the 15th onto the 14th, and so on, until within perhaps fifteen seconds the entire east side of the structure had collapsed into the basement and sub-basement. Figure 7 shows the collapse in progress and Figure 8 shows the aftermath of the collapse.

The Commission interviewed twelve eyewitnesses and the accounts of several others were made available. Since there was no significant disagreement among these several accounts, the Commission has not interviewed each and every eyewitness. While other eyewitnesses may be able to elaborate in more detail or specificity on certain aspects, the Commission believes that the essential events would not be in dispute.

The following account tells the story of the tragedy as seen by eyewitnesses. While the account appears as statements of fact, it must be understood that it is only the Commission's interpretation of testimony. The Commission recognizes that no such interpretation can properly account for the horror of the moment.

As an aid to the narrative, Figures 9 and 10 give the Commission's understanding of the general location of each individual eyewitness and the paths which each followed. Since several eyewitnesses provided helpful information with regard to both the first and second phases, these two phases are combined in the description.

A. PHASE ONE: SHEAR FAILURE IN THE MAIN ROOF AT COLUMN E5 AND PHASE TWO: COLLAPSE OF THE ROOF SLAB

Concrete placement in the mechanical room floor slab, walls, wall beams, and brackets commenced January 25th, at about 10:00 a.m., at the west edge and proceeded east in a continuous fashion. The 5 feet 2 inches high walls, wall beams, and brackets were cast concurrently with the floor slab. A coffee break occurred at about 3:00 p.m. after the concreting had extended about three feet east of line 5. This extent was established by Messrs. Daniel Niro and Joseph Oliva who were employed as cement finishers.

With the exception of Messrs. Niro and Oliva, all those engaged in the concreting went down onto the south side of the roof slab for a coffee break. Messrs. Niro and Oliva remained on the pouring level at about line 4-1/2. Just after the coffee break ended at about 3:25 p.m. they felt a drop in the mechanical room floor of about 1 inch.

Mr. Anthony Paolini, the labor foreman, was standing on the south side of the mechanical room floor. He was directing the crane to move in the next bucket of concrete when he felt the same settlement. In a few seconds another drop of 2 to 3 inches occurred. He shouted instructions to "hold the bucket" after a brief exchange with Mr. Antonio M. Fantasia, the carpenter foreman.

Mr. Fantasia was on the south side of the roof slab. He had noticed a settlement of the roof slab. After dispensing with the thought that the formwork for the mechanical floor had failed, he and Mr. Peter Marino immediately went to the 16th floor by way of a ladder in the east stair opening. After pausing at the stair opening he proceeded directly to column E5 on the 16th floor. As he stated it, "I can't believe my eyes. I see this slab coming down around the column." Mr. Fantasia estimated a drop of 5 to 6 inches at this time and described a crack in the bottom of the slab extending from column E5 toward column D8. With no hesitation he immediately screamed a warning to the workmen on the 16th floor that the roof was going to collapse, ran back to the east stair opening, climbed up the ladder, screamed a warning to the men on the top, and then climbed down to the 15th floor where he and several others remained on an east balcony until the collapse of the roof slab occurred. (In subsequent private testimony Mr. Marino, an electrician who was placing electrical conduits in the mechanical room floor slab, confirmed Mr. Fantasia's description of these events.)

During the interval Messrs. Niro, Oliva, and Paolini, as well as the others on the mechanical floor, had gone down onto the roof slab. Mr. Niro had called

the attention of his foreman, Mr. Peter Molle, who went up onto the mechanical room floor to observe the sag. The sag must have increased rapidly because Mr. Molle got trapped in it and later had to be rescued by the crane.

The Commission interviewed six of the men who were working on the 16th floor: Mr. Paul A. Scopa, the bricklayer foreman; Messrs. Alfred Coco, Christopher Feudo, Kenneth Fielding, and Albert Fiorilli, all bricklayers; and Mr. Edward J. Amerault, a steamfitter.

The distressed area around column E5 was specifically pointed out to Mr. Scopa by Mr. Fantasia. Mr. Scopa went down the east stairwell ladder to the 15th floor, went across to the west side, and ran down 15 flights to the ground, It is of interest to note that the Structural Subcontractor was in the process of placing reinforcing steel for the stairs from the 14th to 15th floors in the east stairwell. This may account for the fact that several of those working on the east side of the 16th floor or roof stopped temporarily at the 15th floor and/or subsequently crossed over to the west stairwell before descending to the ground.

Mr. Coco did not notice the sag of the roof slab. He heard Mr. Fantasia's warning and a second by Mr. Marino. After he saw everybody running he went to the 15th floor via the ladder in the east stair opening. There he; along with Mr. Feudo, Mr. Fiorilli, and Mr. Frank Brangiforte, took refuge on the overhanging swing scaffold in the middle of the east edge of the building. (This scaffold is evident in Figure 6a.) As he stated it, "we stood there for a couple of minutes, and then the roof let go, and I saw everything started to shake, so we jumped from the scaffold onto the balcony. That's when the whole roof collapsed, and then it stopped." Mr Coco then went across the 15th floor to the west stairwell and descended to the ground.

Mr. Feudo was working in the southeast corner of the 16th floor. He heard Mr. Fantasia's warning and noticed the sag. He described it as a belly, about as wide as a large conference table, extending from the southeast corner of the elevator core toward column E5. He followed the same exit path as Mr. Coco.

Mr. Fielding was working on column C7 from a swing scaffold. He did not hear Mr. Fantasia's warning and never observed the initial failure. He was alerted by Mr. Coco's fast exit and after climbing up from his work platform he went to the west stairwell. He stated that he was on the 14th floor when the roof slab collapse occurred and somehow arrived at the ground after Messrs. Coco and Feudo.

In his testimony Mr. Fielding mentioned hearing cracking noises about 1:00 p.m. while working on column C7. He related the noises to the sound of ice cracking. As he stated it, "I do a lot of ice-fishing, and we heard the noise, and my boss said to me 'Holy mackerel, what kind of noise is that.' Approximately 15 to 20 minutes after that, the same noise." Although Mr. Fielding dismissed these sounds at the time, it is quite possible that they were emanating from the roof slab under the west side of the mechanical room floor. Subsequent examination has revealed load-induced cracks over the tops of columns E8, E9, F8, and F9 where failure did not occur.

Mr. Fiorilli was working on the northeast corner of the 16th floor. By the time he reached the stairwell the sag had increased to 4 feet. He went to the 15th floor with Messrs. Feudo and Coco. After the east half of the roof slab had collapsed onto the 16th floor he climbed back up the ladder in the east stairwell to see what had happened. He saw that the roof slab was sloping down to the 16th floor and smelled gas. Fearful of an explosion, he backed down, crossed the 15th floor, and ran down the west stairs with Messrs. Coco and Feudo.

Mr. Amerault was on the north side of the 16th floor hanging a length of pipe. He heard Mr. Fantasia scream, saw the sag, and went down the east stair opening. He got as far as the 14th (15th?) floor when the roof slab collapsed. After the collapse he went across to the west stairs, went back up to the 16th floor, and walked back toward the elevator shaft. On looking he could see the roof slab slanting down and lying on his pipe machine. Like Mr. Fiorilli, he smelled the propane gas and was afraid of an explosion. He ran down the stairs as fast as he could.

Mr. Di Simone, the steel foreman, was in the office of the Structural Subcontractor when the crane oiler told him that something had gone wrong. He went to the crane intercom and talked with Mr. Paolini. He immediately went to the construction elevator and started up. As he rode up, the roof slab collapsed. He saw the compressor falling off the roof along with other debris and dove out at the 8th floor. Some other workmen told him to clear out so he went down the west stairs to the ground and then back to the south side of the building.

While talking with Mr. DiSimone, Mr. Paolini could see, slowly but surely, that the whole bay E5-E6-F6-F5 was sinking in. He could hear creaking and groaning and saw the reinforcing steel popping up out of the mechanical floor slab. He went to the west stairwell with other workmen but found it covered by canvases. One man jumped right through, but Mr. Paolini and the others went back to call for the construction elevator (materials hoist).

The compressor which had fallen from the roof landed on top of the mason-tender's shack. Mr. Robert G. Delaney, the operator of the materials hoist, checked to make sure that the mason-tender had not been crushed and then returned to the hoist. He directed the hoist from the 8th floor up to one of the top floors of the building. As he then brought the hoist down to the ground, several men got on at various levels.

Mr. Paolini and the others were still on the roof calling for the hoist. Mr. William Murphy, a foreman for Dorel Steel Corporation, got on the hoist and rode directly up to the roof. Messrs. Paolini, Niro, Oliva, and about six other workmen got on with Mr. Murphy. Mr. Delaney brought them directly to the ground. Before descending, Mr. Paolini recalls looking back and noting that only one of two boilers was still visible. These boilers had been temporarily stored on the roof slab for about two weeks in approximately the locations shown of Figure 9.

Three men - Mr. Molle; Mr. Patrick McNulty, an ironworker; and a carpenter - were marooned on top of the collapsed roof slab.

When Mr. Di Simone arrived again in the back of the building he looked up and saw Mr. Paolini and the others on the roof waiting for the elevator. He recalls hearing sirens at that time and thinking that one of the concrete truck drivers had called the police. In fact, Mr. Pierre Fontaine, the job superintendent for the General Contractor, had called them. He had been in his office and saw the compressor falling in the vicinity of line 5. He went to his office door and talked with a bricklayer. Together they called the police and asked for ambulances.

In any case, Mr. Di Simone waited for the group of men to descend from the roof. In order to assist in rescuing the three marooned workers and fearing that some other workers may have been trapped in the 16th floor, Messrs. Murphy and Di Simone encouraged Mr. Delaney to take them back up to the roof. According to Mr. Murphy, the hoist stopped at the 16th floor on this trip. Mr. Murphy climbed up to the top of the hoist staging and from there directed the rescue of the three marooned men.

Mr. Mark Seaver, the crane operator, had emptied the concrete bucket and Mr. Murphy, by the use of hand signals, directed Mr. Seaver to swing the bucket around to pick up the three men. Mr. Di Simone, on the 16th floor, looked out through the rubble as best he could and after a few minutes decided that no one had been trapped. Mr. Di Simone felt some movement in the 16th floor slab at this time and Mr. Murphy could hear popping and cracking as well as see movement in the roof slab from his position on the hoist staging.

Once the three marooned men had been picked up, Mr. Murphy climbed back down to the 16th floor. Satisfied, Messrs. Murphy and Di Simone descended to the ground in the hoist. Mr. Di Simone returned to the office, but Messrs. Murphy and Delaney remained by the materials hoist. Within a minute or so the general collapse occurred. As they ran away Mr. Delaney fell down and was hit on the arm by something. His wrist was broken.

B. PHASE THREE: THE GENERAL COLLAPSE

The general collapse is clearly and succinctly described in a written statement by Mr. Douglass N. Ellis, Jr., a resident of 1959 Commonwealth Avenue. The photograph in Figure 7 was taken by Mr. Ellis.

"I stepped outside and noticed the top floors - one or two - had collapsed, and sent debris flying out and down. At the same time, I raised the camera and began to take pictures. I took three while the structure was falling. They were taken in the minimum time it takes to trigger the shutter and advance the film. The subsequent views I had of the building falling were through the camera lens.

"As the top floors fell, there was not much debris above the collapse, but considerable dust and debris below the collapse and blowing out from the floors that were actually in the process of collapsing. The collapse progressed in what is best described as domino fashion. That is, the weight of what had already collapsed seemed to blend into where a floor was, then this floor would fall, still somewhat defined by the balcony rail; then as that floor reached the next, the debris from the floor above would become undefinable.

"By the time one-half to three-fifths of the side had collapsed so much dust and debris had risen from the ground and been squirted out from between the collapsing floors that accurate observation of the progression of the fall had become difficult.

"However, the sense of movement continued in the dust cloud and it seemed that around the building the dust and debris was sucked downward and only rising into the air from the outer edges of the collapse.

"While the fall was in progress, some pieces of debris of discernable size and shape were visible, such as wooden supports and what looked like window frames. The building did not seem to fall at once, but in a floor by floor progression, nor did it appear to fall away from the core that remained after the collapse.

"But as each floor fell, it fell completely in that nothing remained around the central core at the level where that floor had been. That is, there was no major collapse with a series of additional pieces falling even seconds later.

"All that remained was the central core of that end of the building and broken and torn pieces of reinforced concrete flooring that hung from the core of the center of the building."

As may be seen in the photograph taken by Mr. Ellis (Figure 7), failure of a given floor in the vicinity of lines 6 and 5 was in advance of failure at line 2. This fact would indicate the likelihood that the first failure in the 16th floor occurred at line 5 or at the elevator core. In fact, one of the boilers, which had been on the south side of the main roof near line 6 was found well down in the debris.

The photographs in Figure 8 show the extent of the collapse and Figure 11 details it. About 120,000 square feet of floor slab collapsed into the basements and lay like pancakes. Another dimension of the collapse may be obtained when it is considered that about 8000 tons of construction materials fell to the ground.

Four men were crushed to death in the general collapse. The search for their bodies was protracted because of the necessity to remove the many thousands of tons of entangled reinforced concrete debris. Mr. Ciriaco Di Iorio, a laborer for Pizzotti Brothers, Inc., was found in the area of the east stairwell on February 1, 1971. Mr. Scopa, Pizzotti's bricklayer foreman, testified that Mr. Di Iorio and Mr. Luigi Casale went down the east stairwell. At the 3rd floor Mr. Casale ran across to the elevator core to get his coat, just making it to the west side when the general collapse occurred. The other three men, Messrs. James Cingolani, Michael Papasedero, and Daniel Tintindo, were employed by Loustan Contracting Corporation. All three were found together on February 15, 1971, in the basement area just southeast of the elevator core. Most of the 8000 or so tons of debris had to be removed before they were found.

3.

THE CAUSE OF THE FAILURE

A. PHASE ONE: SHEAR FAILURE IN THE MAIN ROOF AT COLUMN E5

The crucial aspect of the initial failure occurred when the roof slab failed in shear around column E5, although this failure was likely preceded and aggravated by flexural yielding in the roof slab adjacent to the east face of the elevator core.

The Commission has examined the failure at column E5 from the following points of view: (1) whether failure would have been expected if the structure had been constructed in accordance with the design documents (structural plans S-1 through S-9 and the designers' specifications Division 3, Section 3A which are included in Part II, Appendix II. 4); (2) whether the construction procedures and materials conformed with the design documents; and (3) whether the design, as indicated by the documents, met building code requirements. It is the Commission's opinion that:

1. Shear failure around column E5 would not have occurred if the construction had conformed to the design documents.

2. The construction procedures and materials did not conform with the design documents. The most significant non-conformities were:

a. The general lack of adequate shoring under the roof slab in the area bounded by the mechanical room floor slab. As a consequence, the roof slab was subjected to a temporary construction overload by the weight of the freshly placed mechanical floor slab and supporting walls and brackets, their associated formwork, two boilers, and other construction equipment and materials.

b. The fact that the concrete strength in the roof slab was less than specified. The design documents specify that the (ultimate) compressive strength at age 28 days shall be 3000 psi. At the time of the failure, at least 47 days after casting, it was substantially below 3000 pounds per square inch.

The Commission believes that these two factors were the principal causes of the initial failure.

3. The design did not entirely conform with the building code requirements. In the context of the initial failure, the inconsistency which turned out to be most significant pertained to the slab thickness being thinner at column E5 than required. Although the thickness requirements are related to deflection and not strength considerations, a thicker slab would have had a greater shear capacity.

Because some previous shear failures of this type have been attributed to the presence of large openings in a slab adjacent to a column, it should be noted that there is no evidence in this case which would point to the presence of any opening adjacent to column E5.

Shoring

Although structural plans S-1 and S-8 noted that construction loading on the (unsupported) roof slab should not exceed 30 pounds per square foot on the area bounded by the mechanical room wall beams, the weight of the freshly placed mechanical room floor slab and its immediate formwork alone approximated 130 pounds per square foot. Additionally, structural plan S-8 specifically called for the placement of shores under the roof slab below the cantilever brackets which extended outward along column lines 4, 5, 8, and 9 from columns E4, E5, E8, E9, F4, F5, F8, and F9. These shores were to assist in the distribution of the construction loads induced during and after placement of the mechanical room walls and floor slab.

Inspection of the shoring provided under the uncollapsed western half of the roof slab revealed the existence of only five shores under the area of the mechanical room floor slab. One of these was immediately adjacent to the southwest corner of the elevator core wall. The other four, between lines 7 and 8, were used to support part of a cloth wind screen. Testimony indicated that this screen was used to terminate a perimeter enclosure of the east half of the 16th floor. None of the specifically required shores were present under the western half of the roof slab. A plan showing the location of all shores under the uncollapsed west side of the roof slab is contained in Part II, Section 7.

The Commission heard testimony from several witnesses concerning the shoring provided under the collapsed eastern half of the roof slab. It is the Commission's understanding that there was a series of shores along the perimeter of the 16th floor slab extending from the 16th floor slab up to the roof slab. These shores were used to support a weather protection screen for the workmen on that part of the 16th floor.

Messrs. Feudo, Fiorilli and Scopa all testified that they observed the sag of the roof slab after the initial failure at column E5. None of them saw any shoring in the area of the sag. Mr. Fantasia, who first observed the failure, cannot recall whether there was any shoring or not, but he did indicate that he was able to run from column E5 back to the east stairwell without difficulty.

The Commission heard opinions from six eyewitnesses regarding the shoring present on the day of the failure under other interior portions of the eastern half of the roof slab. Messrs. Fielding and Scopa did not recall seeing any interior shores. Mr. Scopa, the bricklayer foreman, stated that there were no interior shores at the time the bricklayers began work in that area, several days before the collpase. Messrs. Amerault, Coco, Feudo, and Fiorilli all thought there were some interior shores, but could not be specific. Mr. Fiorilli referred to them as being "here and there" and Mr. Amerault as being "scattered inside the exact locations and amount I couldn't tell you". Mr. Feudo said that workmen moved about easily, even with wheelbarrows.

Structural Analysis

To assist in determining the cause of the failure, structural analyses of both the east and west halves of the roof slab were made. These analyses consisted of two basic parts: (l) a determination, by a computer analysis, of the axial forces, shear forces, and bending moments induced in the slab and its supporting columns and walls as a consequence of the roof slab dead weight plus the superimposed construction loads and (2) a determination of the capacity of a particular portion of the slab to resist the previously determined forces. These analyses and the results obtained therefrom are described in detail in Part II, Section l. Only the summary conclusions are noted here.

The first computer analyses for the east half of the main roof were made to determine the most highly stressed locations and to establish the influence of perimeter shoring. These analyses indicated that the critical shear conditions did indeed occur around column E5 and, further, that perimeter shoring would have provided no relief for the roof slab in the vicinity of the interior columns. The forces and bending moments at column E5 were only slightly more severe than those at column F5, the difference being due to the fact that two boilers and some other construction equipment were stored on the south side of the roof.

For the vertical shear force and bending moment transfer at column E5 as calculated in these initial elastic analyses, a maximum shear stress of about 180 psi is indicated for a peripheral section of the slab out three inches from the face of the column . With these conditions one might expect a shear failure to occur if the compressive strength of the concrete was below 2000 psi.

The same initial computer analyses indicated the existence of slab bending moments adjacent to the east face of the elevator core which could not have been resisted elastically by the slab as designed. It is the Commission's opinion that shear failure at column E5 was preceded by flexural yielding adjacent to the east face of the elevator core. Such yielding would not have been precipitous in and of itself. It would have evidenced itself by cracking in the top of the roof slab and could not have been noticed because of the formwork enclosure for the mechanical room floor slab and wall beams.

Assuming that flexural yielding did occur, the conditions of the finite element analysis were modified to permit rotation of the roof slab perpendicular to the elevator core. The results incorporating this modification indicate a slight increase in vertical load transmitted to column E5 and a larger increase in

the bending moment transferred into column E5. For the slab to transmit these increased forces, the maximum shear stress in the slab on the same critical perpheral section would increase from the value of about 180 psi to about 205 psi. For this more severe condition, shear failure around column E5 might be expected if the concrete compressive strength was below 2600 psi.

Concrete Strength

The roof slab was cast in two parts. On December 3, 1970, concreting began at the west edge of the roof and proceeded east in a continuous fashion to a construction joint. On December 9, 1970, the remaining portion was completed, beginning at the construction joint and continuing to the east edge of the roof. The location of the construction joint between two castings is of some interest because tests have indicated that concrete from the December 9 placement had somewhat lower strength on the day of the collapse than that in the December 3 placement.

According to testimony of representatives of the Structural Subcontractor, the construction joint between the two castings started at line 3 1/2 on the south edge of the roof, extended north to line D 1/2, then west to line 4 1/2, then north to line F 1/2, then east to line 3 1/2, and finally, north to the north edge of the roof.

The invoices and delivery slips of the ready-mix Concrete Supplier indicate deliveries of 218 cubic yards on December 3 and 95 cubic yards on December 9. These records are inconsistent with the testimony of the Structural Subcontractor's representatives; for while the total delivered yardage is consistent with the calculated volume in the entire roof, the 218 - 95 breakdown indicates that the construction joint was in the vicinity of line 5 and not line 4 as indicated by the testimony.

Clearly, the joint was not where the Structural Subcontractor indicated it to be. Lacking any other direct evidence, it seems likely that the joint was arranged as indicated by the Structural Subcontractor, but shifted one bay to the west. This assumed position, which is shown in Figure 5, is consistent with the yardage breakdown of the two concrete deliveries. With this location the concrete around column E5 is from the December 9 casting.

There are other possible joint locations which would be consistent with the concrete delivery records. For example, the joint could have been directly over line 5, however, the Structural Subcontractor has testified that slab construction joints were always placed at mid-span (either in the zig-zag fashion described above or in a straight line). Two construction joints on typical floors of the uncollapsed west part of the structure are at mid-span.

Another possible and consistent joint location is a zig-zag pattern in which the north-south part of the joint is at line 5 1/2 on the exterior and at line 4 1/2 on the interior. This pattern would imply that the concrete around column E5 was placed on December 3. This possible location is felt to be less likely than the one shown in Figure 5 because it has an even greater discrepancy with the testimony.

Since it has not been established as absolute fact whether the concrete around column E5 came from the December 9 or December 3, 1970, placements, test results regarding compressive strength for both placements are reported in Tables 1 and 2, respectively. Specimens from the December 3 placement were taken from the uncollapsed portion of the roof slab and from one roof slab piece recovered from the collapse. This piece (marked π15) came from bay 6-7 south of the elevator core. All of the December 9 placement was included

in the collapse. Specimens from this placement were obtained from three saved pieces (marked #9, #11, and #20) which were positively identified as coming from the southeast corner bay of the roof slab.

Based upon an evaluation of the compressive strength test results in Tables 1 and 2, which is contained in Part II, Section 2, it is the Commission's opinion that the average compressive strength, at the time of collapse, was approximately 1600 psi for concrete placed on December 9, 1970 and was approximately 1900 psi for concrete placed on December 3, 1970. At the time of collapse the ages of the two placements were 47 and 53 days, respectively.

The deficient roof slab concrete strength might have resulted if the concrete deposited in the forms never had the potential to reach the specified strength, or if the deposited concrete was improperly cured, or both. As indicated in the following discussion, it is the Commission's opinion that both of these possibilities were factors.

The potential strength of a given batch of concrete could be low because of one or more factors. The most common reasons would be a low cement content and/or a high water content, but still other factors such as the amount of entrained air will influence the potential.

Concrete mix designs for this project were submitted by the Concrete Supplier to the General Contractor. According to the Supplier's delivery slips the 3000 psi concrete contained 540 pounds of cement per cubic yard of concrete. It is the Commission's opinion that this mix design cement content is passable for 3000 psi concrete if the water content is carefully controlled. Although the design specifications require air-entrainment for the exposed portions of the roof slab, the delivery slips do not indicate the inclusion of such an admixture for any part of this slab.

To determine the actual cement content of the roof slab concrete, chemical analyses were made of five specimens from the December 9 placement and six specimens from the December 3 placement. The results of these analyses are also tabulated in Tables 1 and 2. For concrete in the December 9 placement the average of the five cement content determinations is 501 pounds per cubic yard, and for concrete in the December 3 placement the average of six determinations is 535 pounds per cubic yard.

With regard to water content, such site inspection reports as are available indicate that slump test results consistently exceeded, during the course of construction, the maximum permissible values specified on the design documents. For the December 9 placement there was no site inspection. For the December 3 placement reported slumps are within the limits specified on the design documents.

Evaluation of the curing conditions while the concrete was hardening with age requires consideration of the temperature history of the concrete. Section 3A of the designers' specifications outlines a detailed procedure for the application of supplementary heat in the event that the ambient temperature might fall below 40°F. within a period of seven days after casting. Testimony heard by the Commission indicated that the only cold weather protection provided for the roof slab was such as may incidentally have occurred as a consequence of the daytime heat provided for the mortar of the bricklayers who were working on the 16th floor in January.

The Weather Bureau records for the casting dates of the roof slab show the following:

	December 3, 1970	December 9, 1970
Maximum	57°F	36°F
Minimum	42°F	21°F
Average	50°F	29°F

The weather turned cold on December 4,1970, and remained cold. From December 4,1970, to the day of collapse the minimum daily temperature never went above freezing. A measure of curing temperatures can be obtained by using the Weather Bureau data for degree days. These records indicate an average curing temperature, from the day of placement to the day of collapse, of about 25°F. for both the December 3 and December 9 placements.

In addition to the previous comments regarding the concrete mix design, cement content, and water content, the Commission believes that these curing conditions seriously retarded the strength gained by the deposited roof slab concrete at the time of collapse.

Design

The structural frame was designed in accordance with the "ultimate strength" method contained in the American Concrete Institute 318-63 Code. For the design conditions used in conjunction with this method, the floor slab thickness in the vicinity of column E5 should have been at least 8-3/4 inches instead of the specified 7 1/2 inches.

The minimum thickness specification in ACI 318-63 is intended to control the vertical deflection of the slab under service live load and is not based on strength considerations. Nevertheless, the design slab thickness did not conform and there is no question that a thicker slab would have possessed a higher shear capacity.

Why Failure Did Not Occur On The West Side

Separate structural analyses were made for the western, uncollapsed, half of the roof slab. These analyses took into account the line of interior shores present in bay 7-8.

The presence of the line of shores kept the slab bending moments at the west face of the elevator core to values approximately one-half those calculated at the east face of the elevator core. Even though flexural yielding did not occur along the west side of the elevator core, the larger superimposed load on the west side created shear conditions around interior columns E8, E9, F8, and F9 which were as severe as those around columns E5 and F5. Observation of the roof slab over the top of columns E8 and F8 reveals crack patterns with widths of up to 1/8 inch and more. While there are cracks over columns E9 and F9, they are not large.

Although failure did not occur on the west side, it is the Commission's opinion that such was narrowly averted. Shear failures occur suddenly and without warning.

The fact that failure did not occur on the west side could be rationalized solely on the basis of statistical strength variations commonly observed in nominally identical laboratory shear tests. These laboratory variations occur

because of unintentional differences in factors such as slab thickness, position of the reinforcing bars, and concrete strength. Each of these factors may have been more favorable on the west side than at column E5. As discussed previously, there is good reason to believe that the concrete strength on the west side was higher than at column E5, and it is a fact that there is no construction joint in the roof slab on the west side.

B. PHASE TWO: COLLAPSE OF THE ROOF SLAB

After the initial shear failure around column E5 and sagging in bay E5-E6, failure in the roof slab suddenly propagated laterally to include almost the entire east half of the roof slab. Although the conditions under which an initial shear failure will occur around a single column are reasonably well known, the state of the art does not contain any accepted procedure for analytically determining the extent of lateral failure propagation in the event of shear failure at one column.

Analysis indicates that the roof slab became intolerably overstressed at other locations once it was deprived of support at column E5. Although the precise relationships have not been established, the question of whether or not failure will remain confined to one region or will propagate to include portions far removed from the initial distress depends principally upon loading, span lengths, and quantity and distribution of steel reinforcement.

Whereas factors regarding reinforcing steel are suggested as having only an aggravating influence on the initial failure (e.g., flexural yielding in the slab along the east face of the elevator core), it is the Commission's opinion that they were more significant in relation to the lateral propagation of the roof slab collapse and to the completeness with which the slab sheared through its supports.

Structural Analysis

Another computer analysis, with column support at E5 removed, reveals that the slab bending moments about line E in bay 5-6 would have changed from negative to substantial positive values. As designed there was an interruption in the north-south reinforcing steel in the bottom of the slab at line E, and therefore, no steel to resist the positive bending moments. These conditions account for the flexural crack and sagging which was noted in this area by Mr. Fantasia and other eyewitnesses.

Additionally, the analysis indicates that the shear condition around column F5 and the flexural and shear condition at the southeast corner of the elevator core became such that failure at either or both locations could be predicted; for a peripheral section of the slab out three inches from the face of column F5, the analysis indicates a maximum shear stress of about 320 psi.

In the same way that loss of support at column E5 caused additional load to be tranferred to the adjacent points of support, subsequent failure at column F5 or at the elevator core wall would induce a similar effect. An analysis was made assuming the failure at column F5 and, as expected, the results indicated serious overstress at the elevator core as well as at columns E4 and F4. This propagation continued until the roof slab had failed around columns on lines 3, 4, 5, and 6 and had dropped down onto the 16th floor slab. Photographs indicate that the slab rotated about, but did not sever through, the exterior columns along line 2. Mr. Di Simone (and Mr. Murphy) testified that a similar condition occurred along the line D columns at the south edge.

Deficiencies In Reinforcing Steel and Concrete

Although they are not noted in any way on the structural plans, the Engineer did indicate on his bar placing drawings one detail calling for bent dowels which would tie the top story columns to the roof slab. Such bent dowels would have helped to limit the propagation of failure in the roof slab but they were not clearly and consistently specified by the Engineer, were not scheduled or shipped by the Steel Supplier, and were not inserted by the Structural Subcontractor.

There also is indication of a deficiency of top slab bars passing directly over the top story interior columns. Either the top story column E5 or F5 (piece marked Π 2) with the wall bracket directly above it was saved from the collapse. Examination of this piece indicated that only one east-west and no north-south top slab bars passed over the top of the column. Whereas the presence of slab steel passing through the critical shear area is believed not to influence significantly the loads at which shear fracture in the slab would occur, the presence of such bars (and bent column dowels) would help to prevent total collapse. There is illustration of this fact at column C7 on that portion of the structure which still remains. At several floors the concrete around column C7 has suffered a complete fracture but collapse has not occurred because of dowel action of the several steel bars passing over the column.

In detailing the east-west bottom steel for bay E5-E6-F6-F5 in the roof slab, the Engineer apparently failed to account for the increased span in this bay and for the edge condition at the elevator core. Whereas these bars, according to the ACI 318-63 Code, should have extended at least 6 inches into the elevator core wall, there was no such detail on the structural plans. The specified bars were 49 inches shorter than required to meet this provision

according to a detail drawing contained in the designers' specification. The absence of this steel at the wall decreased the load capacity at this critical connection.

Of the pieces recovered from the collapse, four were positively identified as being from the roof (marked π9, π11, π15, π20). Detailed examination of the quantity and placement of reinforcement in these pieces, as compared to that specified in the structural plans, bar placing drawings, and bar lists, reveal several discrepancies in each of the pieces. While the discrepancies could not be classified as being gross (in some cases more steel is present than is specified), they do indicate that mix-ups in placement did occur generally and they leave open the possibility that major discrepancies could have occurred elsewhere in the roof.

As noted previously there was a construction joint in the roof slab in the vicinity of line 5. Structural plan S-1 and the designers' specifications require a key and additional reinforcing steel for such joints, but testimony indicated that the specified key was not incorporated and that no additional steel was placed.

The previous discussion about the low strength of the roof slab concrete need not be repeated here, but it is important to recognize that the strength deficiency occurred throughout the slab.

C. PHASE THREE: THE GENERAL COLLAPSE

The 16th floor slab was subjected to an even more severe loading than had been imposed on the roof slab. Ignoring any dynamic effects caused by the fall of the roof, the maximum shear stress induced in the 16th floor slab at the critical section around columns E5 and F5 would have been about 260 psi on the assump-

tion that the distribution of superimposed load on the 16th floor was the same as that on the roof slab. This shear stress could have been increased as a consequence of flexural yielding in the 16th floor slab adjacent to the east face of the elevator core. In this case failure could be anticipated if the concrete compressive strength was below about 4200 psi.

Three 4 inch diameter concrete cores were taken from the uncollapsed 16th floor slab on March 13, 1971. These cores were tested on March 15, 1971, and gave compressive strengths of 3350, 3440, and 3710 psi. In addition a series of 4 inch diameter cores was taken of February 10, 1971, from a piece of floor slab saved from the collapse (marked π 71) which is believed to have come from the 16th floor. Three of these cores appeared to be sound. They were tested on February 12, 1971, and had compressive strengths of 3170, 3440, and 3450 psi. The concrete from one of the three cores from the uncollapsed floor slab was analyzed for cement content. This analysis indicated a cement content of 572 pounds per cubic yard compared to the value of 540 shown on the delivery slip.

Guided by the results of the core tests, it is the Commission's opinion that the average compressive strength of the 16th floor slab, at the time of failure,was approximately 3100 psi.

Lateral propagation of the 16th floor collapse, similar to that on the roof slab, occurred under somewhat different loading and reinforcement conditions. Each successive floor was totally unable to support the load being dropped onto it. Failure, story-by-story,right into the ground was the only possible course. Had it not been for the elevator core,it is the Commission's opinion that the collapse would have propagated into the west half of the structure.

Reinforcement

Two particular features pertaining to the quantity and placement of reinforcing steel deserve mention in addition to those already discussed. First, many one story columns separated from the floor slabs during the collapse and were recovered nearly intact. Although structural plan S-6 clearly specified two sets of ties for each vertical column steel splice as well as the normal tie spacing of one set every 12 or 15 inches along the column height, examination of the recovered columns revealed the consistent absence of any ties in the splice regions at the base of the columns. Quite typically the first ties occurred 36 to 40 inches above the base of the column.

Another feature concerns the effectiveness of the connection between the typical floor slabs and the east face of the elevator core walls. The Structural Engineer made the same detailing error in specifying the length of the east-west bottom slab bars for bay E5-E6-F6-F5 in the 3rd to 16th floors as he made in the roof slab. As a consequence, no bars in the bottom of the slab penetrated into the core walls. Additionally, a careful examination of the fracture surface at several floor levels on the east face of the elevator core indicated that as little as one half of the specified east-west top slab steel was actually placed. And finally, many top steel bars which were placed extended only about one or two inches beyond the face of the wall and/or were at or below mid-depth of the slab. As a consequence of this combination of factors there was a definite plane of weakness at the east face of the elevator core.

It is the Commission's opinion that these deficiencies reduced the structure's capacity to resist the propagation of the collapse; however, it believes that the total collapse probably would have occurred even without them.

Design

Reinforced concrete flat slab construction has been widely used for many years, but a failure of such magnitude and extent is perhaps without precedent. On the other hand, shear failures around columns have occurred in the past and have led to collapses which propagated laterally to involve several bays and vertically to involve several stories. This form of construction, even if designed and constructed in conformance with accepted good practice, may be susceptible to such a propagating collapse in the event of a shear failure such as occurred at column E5.

4.

A CATALOG OF IRREGULARITIES AND DEFICIENCIES

Those deficiencies which the Commission believes to have been technically responsible for the collapse have been enumerated in the previous section. In addition to these technical causes, a number of other non-technical and technical irregularities and deficiencies occurred in the development of the project. With a different focus, this entire series of events, which began before the building permit was issued and continued up to the day of the tragedy, are listed below.

A. BUILDING PERMIT

Immediately following the collapse, the Building Commissioner directed his department to assemble all available documents relative to this project. The plans submitted with the latest building permit application have not been found. The Commissioner states that the Building Department's copies were released for microfilming and somehow were lost. The Fire Department furnished the Commission a duplicate set of architectural, structural, and mechanical prints which carry certain approval stamps which will be referred to in this section. The Commissioner provided the permit; the affidavit; an undated set of plans for the redesign of the heating, ventilating, and air-conditioning system which was commissioned by Loustan Contracting Corporation, the mechanical subcontractor; and a 13 page Loustan Corporation

memorandum entitled "Revisions as required by Boston Building Department - Plumbing Division" dated May 12, 1970. Documents relative to mechanical and electrical approvals and certain shop drawings and brochures related to the mechanical work were furnished by the Building Department and by Loustan.

The building permit which authorized the construction at 2000 Commonwealth Avenue was issued in part on the basis of Section 110 (i) of the then applicable Building Code of the City of Boston (Chapter 479 of the Acts of 1938, Commonwealth of Massachusetts, as amended). This Section permits the Building Commissioner to accept, without detailed review, an application "accompanied by plans and computations and an affidavit filed therewith by a competent architect or engineer to the effect that said plans and computations are in accordance with the requirements of this code and of all other pertinent laws and ordinances." This provision must be understood to assess the significance of the following factors:

1. The words "competent architect and engineer" were in all likelihood written in 1938 when Massachusetts had no registration law for either profession. Registration for professional engineers was adopted in 1941 and became mandatory in 1958. Registration for architects was adopted in 1942 and became mandatory in 1957.

   In 1966, Section 54 A, Chapter 143 of the General Laws of the Commonwealth was adopted, which requires that building departments and certain other authorities shall not approve plans, unless they bear the seal of a registered architect or registered professional engineer. Since this is a Massachusetts statute, the reference is clearly to Massachusetts registration and registration in some other state or country would not qualify.

The provisions of Section 54A of Chapter 143 were ignored for this project. We have examined all documents provided by the Building and Fire Departments and have failed to find one which carries any registration stamp.

The architectural plans carry the name of WEBB, ZERAFA, AND MENKES, Architects of Montreal and Toronto. The structural plans carry the names of WEBB, ZERAFA AND MENKES and of M. S. YOLLES & ASSOCIATES, Consulting Structural Engineers. The mechanical and electrical plans carry the names of WEBB, ZERAFA AND MENKES and of BRAIS, FRIGON, HANLEY, BRETT & RINFRET, Consulting Engineers. We have found no evidence that any principal or employee of Webb, Zerafa and Menkes is registered in Massachusetts. M. S. Yolles holds Massachuetts registration 7971, as a professional engineer. We have found no evidence that any principal or employee of Brais, Frigon, Hanley, Brett & Rinfret is registered in Massachusetts.

2. If computations accompanied the permit application, they have not been found. When M. S. Yolles was requested, after the collapse, to furnish his computations, he refused to do so. These calculations would indicate the design assumptions made, particular options excercised, and analyses performed by the Structural Engineer.

3. A sworn affidavit, under the date of August 29, 1969, was filed with the building permit application. It was signed by P. J. Quinn as Vice President of M. S. Yolles and Associates. Mr. Quinn is not registered in Massachusetts. On December 15, 1969, he filed an application for Massachusetts professional engineer's registration which had not been acted upon by the Massachusetts board as of the date of the collapse.

Mr. Quinn's signature is notarized on August 29, 1969, by Victor G. Dragone, a Suffolk County notary. Mr. Dragone's seal is impressed on the document.

A statement regarding this affidavit was furnished to the Commission by the attorneys for M. S. Yolles and Associates. In this statement it is noted that Mr. Quinn acknowledges the signature to be his own, but states that he did not swear the affidavit, did not intend to have it sworn, and did not at any time travel to Boston in connection with the project. Mr. Quinn does acknowledge that he could have given the signed affidavit to Mr. Jack Palevsky in Montreal.

In testimony to the Commission, Mr. Dragone stated that he did notarize the signature and, coincidentally, at 2000 Commonwealth Avenue. He stated that he was with Bartholomew W. Cosentino, who introduced him to a Mr. Quinn. He also stated that he did not confirm Mr. Quinn's identification.

4. The application for the building permit was dated August 27, 1969, and received by the Building Department August 28, 1969. It is signed by Jack Palevsky, 1 Center Plaza, Boston, as Owner or Authorized Agent for the Owner. The signature of Dante F. Montouri, 81 Dean Street, Cochitutate (sic), Massachusetts (with the note "Mass. P. E. Structural") appears on the line normally signed by the Boston Licensed Builder. Toulon Construction, Inc., 4081 St. Catherine W., Montreal, Quebec is typed in as the Contractor.

Mr. Palevsky is a principal of Toulon Construction, Inc. together with Messrs: John Alper and Bernard Palevsky. They also became part owners of record of 2000 Commonwealth Avenue on September 4, 1969.

Mr. Montouri is not a licensed builder; however, the Building Commissioner wrote below Mr. Montouri's signature "Approved 4 Sept. 69 R. R. Thuma, Jr." The Commissioner has stated that he approved Mr. Montouri because, in his judgment, the qualifications of a professional engineer are at least equal to those of a licensed builder.

In testimony to the Commission Mr. Montouri stated that he signed the application for Ken Cosentino, the son of Bartholomew W. Cosentino. He further stated that Mr. Palevsky's signature, the name of Toulon, and all addresses were filled in subsequent to his signing. (He noted that the 81 Dean Street address attributed to him was incorrect and that he had moved from that address several years prior.) Mr. Montouri stated that at the time of his signing he expected his firm, LaMont Construction Co., to be the contractor. Perhaps unknown to him was the fact that an excavation permit dated August 1, 1969, had been issued in the name of Toulon.

Section 120 (h) of the Boston Building Code states that:

> If, for any cause, a person licensed as herein provided shall cease to have charge or control of work ... before such work is finished, the work shall stop until another person duly licensed for the doing of such work has been placed in charge thereof.

After signing the building permit application, Mr. Montouri had no further association with the construction of the project. In this case none of the parties involved, Mr. Montouri, Toulon, the Owners, or the the Building Commissioner, took any action which would have led to a stoppage of the construction.

Mr. Montouri could have terminated his responsibility as Licensed Builder by filing a "sign-off" statement with the Building Department, but he did not file such a statement.

Mr. Palevsky knew that Mr. Montouri was to have no further association with the project. It was incumbent on him to supply, in Mr. Montouri's stead, a licensed builder, but no principal or employee of Toulon Construction, Inc. had a Boston builder's license.

The Building Commissioner testified that he never was aware of the fact that Mr. Montouri was not in charge of construction. As noted in Section 4F, the City's Building Inspector made 35 site inspection visits and he knew that Mr. Pierre Fontaine, and not Mr. Montouri, represented Toulon during construction. Apparently the City's Building Inspector did not know that Mr. Montouri was recorded as being the Licensed Builder, and consequently, he took no action to stop construction.

5. When a building permit is issued on the basis of an affidavit, Section 111 (h) of the Boston Building Code states in part that:

> The commissioner shall require that the architect or engineer who signed the affidavit or made the plans or computations shall supervise such work, be responsible for its conformity with the approved plans, and forthwith upon its completion make and file

> with the commissioner written affidavit that the work has been done in conformity with the approved plans and with the structural provisions of this code. In the event that such architect or engineer is not available a competent person whose qualifications are approved by the commissioner shall be employed by the owner in his stead.

The initial affidavit, signed by P. J. Quinn, makes no direct reference to this section. Mr. Quinn provided no inspection nor have we found evidence that inspection by any other architect or engineer was provided. The City's Building Inspector, in 35 inspection visits, also failed to note the absence of Mr. Quinn. The other party who might have done something to see that the intent of the Code was met was the Owner, but the Owner did not engage a professional to inspect construction.

6. The building permit was issued September 5, 1969. The permit application has two stamps affixed under the date of September 4, 1969, one for zoning and one signed by Mr. Justin O'Farrell, plans supervisor, indicating that the plans had been submitted and that he had examined them. The electrical and mechanical plans were determined by the Building Department as not conforming to the Code. Normally other detailed approvals are stamped on the permit before it is issued.

The Building Commissioner has stated that it is not unusual to issue a permit with the understanding that detailed approvals will be added later and that no work to which these approvals are pertinent will be performed prior to the appropriate approvals.

The building permit and the architectural site plan carry approval stamps for egress dated February 4, 1970, and for sprinklers, plumb-

ing, vertical openings, and gas dated February 17 and 18, 1970. Some approvals were not made until May 20, 1970.

Loustan Contracting Corporation, the subcontractor for plumbing, heating, ventilating, and air-conditioning, employed Joseph Schneider, Boston, to redesign the heating, vetilating, and air-conditioning (HVAC) system. Mr. Schneider holds Massachusetts registration 6733 as a professional engineer. His plans were completed and delivered to Loustan in March 1970. An undated set of these plans, carrying no registration seal, were approved by the Building Department on May 20, 1970. A second set, provided by Loustan, do carry Mr. Schneider's registration seal and include revisions dated May 8, 1970, and December 9, 1970.

Although the Building Department has stated that the mechanical and electrical plans were unacceptable as originally submitted, we have found no additional plans except those of Mr. Schneider. Except for the HVAC system, the Commission has not been able to determine the bases upon which the Building Department's subsequent approvals were made.

7. From statements made to the Commission by the Building Commissioner, Mr. Justin O'Farrell, and Mr. Harold Katz, it is clear that Mr. Katz processed, for the Owners, the zoning change on July 3, 1968, and met on more than one occasion in August 1969 with the Commissioner and other members of the Department in relation to the project at 2000 Commonwealth Avenue.

From December 23, 1968, to at least November 10, 1970, Mr. Katz was an owner of record of the property at 2000 Commonwealth Avenue. From July 15, 1968, until September 1969, Mr. Katz was also special

counsel for the Boston Redevelopment Authority. Mr. Katz has stated that he did nothing more than any counsel for the owner might properly have done, and that he sold his interest in the project in September 1970.

B. STRUCTURAL DESIGN

One of the charges to our Commission was to determine whether or not the structural design for the subject building conforms with the Boston Building Code in force in September 1969. Our answer must be in two parts.

First, the design was based upon the "ultimate strength" method contained in the Americal Concrete Institute 318-63 Code. The Boston Building Code in effect in September 1969 does not contain provisions for the design of reinforced concrete structures according to "ultimate strength" methods.

Second, and on the other hand, authorization for the "ultimate strength" design approach is contained in Bulletin #176 issued by the Building Commissioner on May 2, 1967:

> Subject: Ultimate Strength Design - Structural Analysis and Proportioning of Members.
>
> Pursuant to authority granted under Section 116 (1) of the Boston Building Code, it is hereby determined that the design methods and design stresses set forth in the publication of the American Concrete Institute entitled "Building Code Requirements for Reinforced Concrete" (A.C.I. 318-63 - dated June, 1963) shall be deemed to be in compliance with the requirements of the Boston Building Code, Part 26, Section 2601.

In the last few years most reinforced concrete buildings in Boston have been designed following the provisions of the ACI Code.

As his authority for the bulletin the Commissioner refers to Section 116 (1) of the Code which reads in part:

> Whoever desires to substitute for the materials or methods covered by this code, materials or methods of construction ... not covered thereby, shall present to the commissioner plans, methods of analyses ... or other information substantiating the analysis of the system. ... Such materials or methods of construction shall not be used until after the commissioner has issued regulations fixing the practice to be followed, but no such regulation shall have the effect of altering the working stresses for any material herein mentioned.

In regard to the last clause of the quoted portion of Section 116 (1) it should be noted that the working stresses in both concrete and reinforcing steel are increased in "ultimate strength" design over those working stresses which would result if structures were designed in accordance with the "working stress" method of design as provided for in Section 2601 et seq. of the then applicable Boston Building Code. It is necessary to point out, however, that "ultimate strength" design is a rational method of design and that the ACI Code is a nationally recognized code for the design of reinforced concrete structures. The new Boston Building Code, which took effect on July 1, 1970, directly incorporates the ACI Code.

The Commission's consultants have reviewed in detail those aspects of the structural design which they believe might have contributed to the collapse, including the design of the roof slab, 16th floor slab, and 16th story columns and walls. For these aspects a following list enumerates the design features and details which were found not to conform with the ACI 318-63 Code.

The structural design of the entire structure was also reviewed, although in much less detail. Excepting the enumerated nonconformities, the conception and execution of the overall design was judged to be generally in accord with the ACI Code.

These analytical reviews have been conducted in relation to the available structural plans. It must be repeated that the Structural Engineer's calculations - including the particular options exercised, assumptions made, and analyses performed - have not been made available to the Commission.

1. Around column E5 and at other critical areas in the roof and in the typical floors the design slab thickness was 7 1/2 inches. For the design conditions stated on the structural plans, the ACI Code would require a minimum thickness of 8-3/4 inches for the slabs in bay E5 - E6-F6-F5, and thus, at column E5.

   If the steel reinforcement in bay E5-E6-F6-F5 had been designed for a yield stress of 40,000 pounds per square inch instead of 60,000 pounds per square inch as noted on the drawings, a thickness of 7 1/2 inches would satisfy the ACI Code requirements. For the analyses made by the Commission's consultants, the design top slab reinforcement extending into the elevator core would not satisfy the design conditions, given a steel yield stress of either 40,000 or 60,000 pounds per square inch.

2. There were several discrepancies on the design drawings between the slab openings shown by the Mechanical Engineer and those shown by the Structural Engineer. Since the Structural Engineer used his own plans to detail the reinforcing steel there were inherent conflicts in the design. (The actual openings placed in the slabs did not correspond to either the Structural Engineer's or the Mechanical Engineer's openings.)

3. The east-west bottom steel in bay E5-E5-F6-F5 of the roof and typical floor slabs was detailed incorrectly. To satisfy the ACI Code these bars

should have extended at least 6 inches into the elevator core walls. A detail drawing contained in the specifications provided for this requirement but there was an inconsistency between this detail drawing and the specified bar length. Had the bars been 21 feet 7 inches long instead of the 17 feet 6 inches specified, the intent of the detail drawing and the ACI Code could have been met.

4. The ACI Code requires that at least 25 percent of the negative slab reinforcement in each column strip pass over the column within a distance "d" on either side of the column face. At columns E4, E5, E8, E9, F4, F5, F8, and F9 in the roof slab neither structural plan S-8 nor the bar placing drawings specifically provided for this requirement. A uniform distribution of the specified steel would nearly, but not quite, meet the requirement. This deficiency is evident in a part of the roof slab over either column E5 or F5 which was saved from the collapse (marked π 2).

C. CONTROLLED CONCRETE

Concrete Mix Designs

1. The specifications prepared by the designers state that, prior to beginning work, certificates of inspection and testing, prepared by an approved inspection company, are required as to the compliance of the proposed aggregates and cement with the specifications. Upon careful investigation no evidence has been found by the Commission to indicate that such inspection and testing were performed or that such certificates were submitted.

2. The specifications also required, prior to beginning work, submittal of the "mix proposed for each class of concrete giving proportions by dry weight of cement, coarse and fine aggregate, type and amount of admixture or air entraining agents, and water-cement ratio." This submittal was to contain "certification that compressive strength, slump, entrained air content and other specified properties will be met." Likewise Section 2612 of the Boston Building Code, which applied to this project, required that tests be made in advance of the beginning of construction to establish that the proposed concrete mix designs would adequately provide for the specified concrete quality requirements.

A letter from J. H. McNamara, Inc. to Toulon Construction, Inc., dated November 7, 1969, proposed certain mixes for 3000, 4000, and 5000 psi compressive strength concrete. The submitting of this letter is all that was done relative to concrete mix design. Although the letter did not satisfy the Code or specification requirements, the mix designs proposed therein apparently were the basis upon which J. H. McNamara supplied concrete to the site. The proposed mixes were:

Class Nominal Strength, psi	A 3000	B 4000	D 4000	C 5000
Cement, lbs/cu.yd.	540	580	640	705
Sand, lbs./cu.yd.	1410	1380	1330	1220
Gravel, lbs/cu.yd.	1860	1860	1860	1860
Water, gal.	34.0	35.0	35.5	36.3
Water/Cement Ratio, gal./sack	5.92	5.7	5.26	4.84
Darex, ounces/cu.yd. (air entraining admixture)	1.0	1.0	1.5	2.0
Slump, inches	4	4	4	4

These mixes were constituted with Atlantic and Atlas Type II cement and Plainville (Massachusetts) sand and gravel.

3. In the following respects the proposed mixes do not conform to requirements as stated in the designers' plans and specifications.

   a. The design and the strengths of the proposed mixes were not developed or verified by fabrication and test of a series of laboratory trial mixtures utilizing the specific constituents proposed for use in this project.

   b. The specifications require a water reducing admixture for all concrete, but none was included in the proposed mix designs.

   c. For concrete which will be exposed to the weather, the specifications limit the water/cement ratio to a maximum value of 0.49 by weight (5.5 gal./sack). In many places, such as balconies, parapets, and the overhanging portion of the roof slab, 3000 and 4000 psi concrete is exposed to the weather.

   d. Structural plan S-1 specifies that slump for all concrete shall be 3 plus or minus 1/2 inch.

4. The specifications and the Boston Building Code require that the mix designs be capable of safely providing the indicated compressive strengths at age 28 days. For the proposed mixes the cement content and the water/cement ratios are the most significant parameters which would affect potential 28-day strength under standard conditions of curing. For the four proposed concretes it is the Commission's opinion that the proposed cement contents and water/cement ratios are:

Class Nominal Strength	A 3000 psi	B 4000 psi	D 4000 psi	C 5000 psi
Cement	passable	low	satisfactory	low
Water/Cement Ratio	passable	high	satisfactory	high

J. H. McNamara's proposal indicates that the mix noted by 4000 psi (D) above is intended to achieve a strength of 2000 psi at age 3 days. This mix design may have been proposed to meet certain construction conveniences, for the structural plans and specifications make no reference to different grades of 4000 psi concrete.

The Supplied Concrete

J. H. McNamara's delivery slips (and invoices) indicate that 2500, 3000, 4000, and 5000 psi concrete was delivered to the site. The volumes of these classes of concrete were 105.5, 4780.5, 3495.5, and 517.5 cubic yards, respectively, for a total delivered volume of 8899 cubic yards. Of the 3495.5 cubic yards of 4000 psi concrete, the invoices show 2993.5 cubic yards with a cement content of 580 pounds per cubic yard (as proposed for 4000 psi (B) concrete), 125.0 cubic yards with a cement content of 640 pounds per cubic yard (as proposed for 4000 psi (D) concrete) and 377.0 cubic yards with cement content not stated.

1. The invoices do not indicate a mix for any of the delivered 2500 psi concrete. The lowest specified strength on the structural plans is 3000 psi.

2. The structural plans specify air entrainment for all of the 2nd to 5th floor slabs, for foundation and retaining walls where exposed to the weather, and for all other concrete exposed to the weather (including the 6th floor to the roof slab). The invoices indicate that Darex (an air entraining admixture)was included in the delivery of 1100 cubic yards of 3000 psi concrete, 701 cubic yards of 4000 psi concrete, and 31.5 cubic yards of 5000 psi concrete. These indicated deliveries did not provide for the required air entrainment in several locations, e.g., in the roof slab.

3. Although the designers' specifications prohibited any addition of calcium chloride, the Supplier's invoices indicate that 1 or 2 percent of calcium chloride was added to 1741 cubic yards of the delivered concrete.

4. Although the designers' specifications required a water reducing admixture for all concrete, the Supplier's invoices indicate that such an admixture was included in only 296 cubic yards of concrete delivered on July 22, 1970. This concrete was placed in the 2nd floor slab.

5. The concrete in the roof slab was specified to have a 28-day strength of 3000 psi. It was cast on December 3 and 9, 1970. The Supplier's invoice for December 3 indicates delivery of 218 cubic yards of 3000 psi concrete with no mention of cement content. December 9 invoices indicate delivery of 85 cubic yards of 3000 psi concrete having 540 pounds of cement per cubic yard and 10 cubic yards of 3000 psi concrete with no mention of cement content. Essentially all of this concrete would have been required to complete the roof slab.

   Analysis of actual cement content in six specimens taken from the December 3 casting shows an average of 535 pounds of cement per cubic yard. High

and low values of 596 and 493 pounds per cubic yard, respectively, were obtained.

Analysis of actual cement content in five specimens taken from the December 9 casting shows an average of 501 pounds of cement per cubic yard. High and low values of 530 and 488 pounds per cubic yard, respectively, were obtained.

6. 240 slumps were reported by the Testing Laboratory during the course of construction. The reported slumps ranged from a high of 7 to a low of 3 inches. In only 37 of the 240 cases were the reported slumps equal to or less than the maximum permissible value (3-1/2 inches) in the designers' specifications.

Inspection

Requirements for inspection and testing of concrete are stated in the specifications prepared by the designers as well as in the Boston Building Code.

The designers' specifications call for appointment by the Architect of a testing company to make inspections and tests. Further they require a minimum of one strength test (4 cylinders) for each 100 cubic yards of each class of concrete and not less than one strength test for concrete placed on any one day. A slump test is required to accompany each strength test and an air content test is required for each 50 cubic yards of air entrained concrete. The specifications require the contractor to furnish, at the construction site, a facility for the proper storage of test cylinders.

For all concrete in this project the Boston Building Code required an approved concrete control engineer for:

a. Inspection of concrete proportioning and mixing at the Supplier's plant.

b. Inspection of concrete placement at the construction site.

c. Conduct of a minimum of one strength test (at least 3 cylinders) for each 150 cubic yards of each class of concrete and of not less than one strength test for concrete placed on any one day.

1. For the 132 days on which concrete was delivered and placed, reports of the Testing Laboratory indicate that inspection at the batch plant occurred on only 44 days, inspection of site placement occurred on only 49 days, and strength tests were conducted for only 46 days of concrete placement. Accordingly, the inspection and testing provisions of the Boston Building Code were met on about 35 percent of the days, not 100 percent as required. Inspection and testing occurred relatively more often for the larger deliveries, but even on a concrete volume basis, less than 70 percent of the 8899 cubic yards were inspected and tested.

Concerning the lack of 100 percent inspection and testing, the one field employee of the General Contractor testified that inspection and testing was provided only at his request and that he did not request 100 percent inspection and testing.

In testimony, a representative of the Testing Laboratory acknowledged that the Laboratory provided service only when requested to do so. Also in testimony, a representative of the Supplier acknowledged that he knew that 100 percent inspection was not being provided on the concrete shipped to 2000 Commonwealth Avenue. Further, he testified that the

lack of 100 percent inspection was not uncommon for other projects in Boston for which his firm supplied concrete. Although both the Testing Laboratory and the Supplier knew that provisions of the Boston Building Code were being violated neither of them demonstrated any objection.

2. When a set of cylinders was taken for compressive strength testing only three cylinders were fabricated, not four as required in the designers' specifications. In several instances the Testing Laboratory reports contain strength test results for only one of the three cylinders in a given set, the reported result being for a 7-day strength test and the missing results being for 28-day strength tests.

3. Testing Laboratory reports contain 28-day compressive strength results for 111 cylinders. The reported results are summarized below:

Nominal Strength (psi)	3000	4000	5000
Number of tests	35	52	24
Highest strength (psi)	4238	4560	5689
Average strength (psi)	3388	4097	5107
Lowest strength (psi)	2798	3080	4481
Coefficient of Variation (percent)	9	6	5

As indicated by the coefficients of variation, the scatter of these reported test results is unusually low. Such consistency was not obtained for tests of the relatively few concrete cores which subsequently were taken from the uncollapsed portion of the structure. By comparison, 28-day compressive strength results for concrete whose quality is excellently controlled usually will have a coefficient of variation of up to 10.

Another measure of the uniformity of the reported laboratory test results is obtained by noting the difference between test results of companion cylinders. The 111 28-day cylinder test results were obtained from 54 pairs and 1 set of three companion cylinders. For these 55 groups the maximum difference in reported compressive strength among a given group is 297 psi. Such uniformity is unusually good for field specimens.

4. Included in the reported 28-day strength test results for the 3000, 4000, and 5000 psi concretes are 2, 10, and 6 cylinders, respectively, which fall below the specified strength values. There is no evidence to indicate that the Testing Laboratory made any remonstration concerning these low results other than to report them on the Laboratory reports.

5. Not one of the Testing Laboratory's 44 plant inspection reports make remonstration against the fact that mixtures batched deviated not only from specification requirements but also from the proposed mix designs.

6. In only one of the Testing Laboratory's 49 concrete placement inspection reports is there any indication that the inspector made any remonstration concerning excessive water contents. This, in spite of the fact that results of slump tests performed by the Laboratory consistently exceeded not only the maximum permissible values cited in the specifications but also the values noted in the proposed mix designs.

7. Although required by the designers' specifications no air content tests were performed.

8. Although required by the designers' specifications (and by common practice) no proper site facility for the temporary storage of concrete test cylinders was provided. After the building collapse, a number of such cylinders were found in the ice and snow in various places on the property around the structure.

Curing

The specifications prepared by the designers state that all concrete surfaces must be maintained at a minimum temperature of 55°F and in a moist condition for seven days after placing. The specifications prescribe that "when outside temperature falls or may fall below 25°F but not below 10°F during placing or during the protection period, provide adequate enclosures of concrete work with tarpaulins or insulation. Provide supplementary heat."

The roof slab was cast partly on December 3 and partly on December 9, 1970. The specified protection periods extended to December 10 for the December 3 placement and to December 16 for the December 9 placement. Official U. S. Weather Bureau records for Logan Airport, Boston, indicate minimum daily temperatures ranging from 15 to 24°F for both the period December 4 to December 10 and the period December 10 to December 16, 1970. Inasmuch as inland temperatures in Boston normally are below those at Logan Airport during periods of cold weather, the minimum temperatures at 2000 Commonwealth Avenue likely were lower than those in the Weather Bureau records.

1. Testimony has indicated that no enclosures were used to protect the roof slab during the specified protection period, nor was supplementary heat or moisture purposefully provided for its protection.

As indicated in Section 1D, the specified protection was excluded from the Structural Subcontractor's contract. This responsibility was assumed by the General Contractor.

2. Because of its crucial bearing upon the collapse, cold weather protection for the roof slab is emphasized above. In a more general sense, testimony has indicated that the only purposeful cold weather protection which was provided during the two wintertime construction periods occurred on one or two occasions in the winter of 1969-1970. For example, no purposeful protection was provided for any concrete in the thirteen deliveries between December 10, 1970, and January 21, 1971, yet the minimum daily temperature at Logan Airport ranged between 2 and 31°F during that period.

Concrete Strength

Based upon an evaluation of test results obtained from concrete cores taken from the uncollapsed portion of the structure and from certain pieces saved from the collapse, it is the Commission's opinion that the in situ compressive strength at 28 days was below the 3000 psi specified by the structural plans in the following locations:

1. The entire main roof slab. In fact, the roof slab concrete strength was deficient at the time of collapse some 47 and 53 days after placement.

2. The entire 16th floor slab.

3. The top story columns (16th floor to roof).

4. The west (uncollapsed) side of the 15th floor slab.

Although not a factor at the time of collapse, tests indicate that concrete in the mechanical room (penthouse) floor slab had not reached the specified 3000 psi strength on March 15, 1971, some 49 days after placement.

D. REINFORCING STEEL

1. Although the structural plans, specifications, and reinforcing bar lists clearly stated that all reinforcing steel should be new billet steel, a substantial percentage of the straight bars supplied for both columns and slabs was rail steel. The Steel Supplier has testified that he had only an oral contract with the Structural Subcontractor to supply high-strength steel with no mention of billet or rail steel and that he did not bid on the basis of meeting specification requirements. Nevertheless, the bar lists from which the Supplier filled the orders clearly noted that all steel should be new billet steel.

   Nine specimens of rail steel reinforcement taken from collapsed pieces were tested at room temperature. For eight of these ultimate elongation was obtained. While the ultimate elongations generally were significantly less than obtained from specimens of billet steel, only two specimens failed to meet the American Society for Testing and Materials' minimum requirements for new billet steel.

2. In addition to the rail steel manufactured by the Supplier, billet steel reinforcement manufactured in the United States (by Bethlehem Steel Corporation, Hawaiian Western Steel Limited, Milton Manufacturing Company and United States Steel Corporation) as well as in Argentina and Taiwan was found in the structure.

Fifteen specimens of billet steel reinforcement taken from collapsed pieces were tested. One specimen had a very low ultimate elongation and another just failed to meet the yield strength requirements.

3. The designers' specifications require a series of tests for each 100 tons of steel. No such tests were conducted and according to the Supplier were not part of his oral contract.

4. There were many deficiencies and deviations in the placement of specified reinforcement. It is, of course, a matter of judgment as to the borderline between deviations which occur in normal good practice and deficiencies which can be considered poor workmanship. While an exhaustive search for all possible deficiencies and deviations was not made, those listed below were recognized in the course of the investigation.

   a. Ties in the bottom 36 to 40 inches of numerous columns were omitted during construction.

   b. Concrete cover for the vertical column bars varied from $\frac{1}{2}$ inch (piece marked π79) to 8 inches (piece marked π19) instead of the specified $1\frac{7}{8}$ inches.

   c. One collapsed column had a cross-section 12 x 29 inches with six #6 vertical steel bars (piece marked π3). All of the 12 x 29 inch columns on structural plan S-6 called for at least eight #6 bars.

   d. The vertical steel in one collapsed exterior column (piece marked π23) consisted of two #7 bars and four #8 bars. There were seven collapsed columns of this size which should have contained six #8

bars and fourteen which should have contained six #7 bars, but none which called for the reinforcement found in piece #23. A similar discrepancy existed in the column piece marked WF2.

e. The vertical bars for the top story columns were detailed and supplied in 9 feet 2 inch lengths. Two top story columns saved from the collapse (marked #2 and #10) had vertical steel over 10 feet long. One bar from column F8 on the west side of the structure protrudes above the top of the roof slab and three of the bars from column G9 have been burned off where they protruded above the top of the roof.

f. The east-west top slab steel running into the elevator core wall in the typical floors was detailed and supplied as four #6 bars and five #4 bars on each side of the hall opening. Detailed inspection of the east face on the 8th through 12th floors revealed a consistent deficiency of steel. No evidence of any #6 slab bars was found. In one case the four #6 bars were replaced with four #4 bars and the five #4 bars were omitted entirely. By appearance of photographs taken with a telephoto lens, these deficiencies were present in all floors. The use of #4 bars in place of the specified #6 bars is evident as well on the west side where failure propagated around the corner of the elevator core on one of the typical floors.

g. The vertical position of the slab reinforcement is erratic. In several instances, the top steel is actually at or below mid-depth of the slab, especially around the elevator core.

h. Four slab pieces saved from the collapse were positively identified

as being from the roof slab. Comparison of the reinforcement in these pieces with that which was detailed and supplied revealed several deficiencies (as well as overages). These are identified in Part II, Section 3.

i. The roof slab and several of the typical floors were placed in two sections. The construction joint between roof slab pours is believed to have been partly at line 4½ and partly at line 5½. Most such construction joints on typical floors were located on the east side, but they do exist at two levels on the west side. Although structural plan S-1 specifies placement of additional steel dowels across such joints, testimony has indicated that no such additional steel was placed.

5. A series of bent bars contained in the Engineer's bar lists for the 16th story columns were not scheduled or shipped by the Supplier. These bars, presumably, were intended to be bent dowels to tie the 16th story columns to the roof slab. Inference of this intent is contained in one detail on the bar placing drawings which called for such dowels. No such bent dowels were placed at the top of the 16th story columns.

The construction joints at the top of the 16th story columns were placed at the bottom of the roof slab instead of 24 inches below the bottom as specified on the same detail drawing which included reference to the bent dowels.

6. Reinforcement was supplied to the job in tagged bundles. Information on the tags included the number of pieces of a particular bar size, the bar length (for straight bars only), and the Supplier's bending designation

mark (for bent bars only). In most cases the Supplier used the same bending designation as shown by the Engineer on the bar placing drawings and bar lists, but occasionally the Supplier designated bent bars with a mark which was used for other straight bars on the bar placing drawings and bar lists. The Commission has attempted, unsuccessfully, to determine the procedures used by the Structural Subcontractor to select and place the reinforcement. One can only speculate as to whether the erratic steel placement was in part caused by confusion in the field with regard to the bar marking.

E. CONSTRUCTION

The lack of adequate shores under the roof slab has been identified as one of the principal causes of the failure. The lack of adequate shores contradicted the Structural Engineer's plans and specifications in the following respects:

1. The specifications state that "the minimum strength of concrete in place for safe removal of soffit forms is 70% of the specified 28 day strength." Further they state "do not strip within one and a half bays of a construction joint until new concrete beyond the construction joint has reached 70% of its specified strength."

   A basis for removing forms and shores from under the west side of the roof slab is the report of the Testing Laboratory which present 7-day compressive strength results for three cylinders from the December 3, 1970, placement. These reported strengths are 2260, 2183, and 2317 psi, each one of which exceeds 70% of the specified 28-day 3000 psi strength. (Note that it is the Commission's opinion that the average *in situ* strength of this concrete, at the time of collapse or age 47 days, was only 1900 psi.)

Since there was no inspection or cylinder testing of the concrete placed to the east of the roof slab construction joint on December 9, 1970, no information was made available to the Structural Subcontractor as to the strength of this concrete. Lacking such specified information, the decision to strip formwork on the east side of the elevator core may have been based upon the general trend of all previously reported 7-day cylinder test results. All but one of these results exceeded 70% of the specified 28-day 3000 psi compressive strength and in this one case the reported strength was 2084 psi.

2. The specifications state that "stripping and reshoring shall proceed simultaneously so as not to leave an area greater than 100 square feet unsupported by either formwork or reshoring at any instant . . . Maintain reshoring in place for 28 days or for such longer time as may be required to ensure that the concrete has reached its designated 28 day strength." It is the Commission's opinion that none of the concrete in the roof slab had reached its designated 28-day strength at the time of collapse.

3. Structural plans S-1 and S-8 clearly noted that superimposed construction loading on the roof slab should not exceed 30 pounds per square foot within the area bounded by the mechanical room wall beams. In this regard the specifications contain a section which states that "the reshores below a completed portion of the structure, which are to support a portion of the structure to be poured, shall be capable of safely supporting the dead load of the structure to be poured plus construction loads." The weight of the freshly placed mechanical room floor slab and its immediate formwork alone approximated 130 pounds per square foot, yet the evidence indicates that adequate reshores were not used.

4. Structural plan S-8 specifically called for the placement of shores under the roof slab below the cantilever brackets which extend outward along column lines 4, 5, 8, and 9 from columns E4, E5, E8, E9, F4, F5, F8, and F9. These shores were to be placed prior to casting of the brackets and mechanical room floor wall beams and were to be capable of carrying 20 kips each. Such shores were not placed next to columns E8, E9, F8, and F9 (the uncollapsed west side) and there is no evidence to indicate that they were placed next to columns E4, E5, F4, and F5 on the east side.

F. CONSTRUCTION CONTROL

In addition to the irregularities and deficiencies regarding inspection and "control" of the concrete, which have been enumerated in Section 4C above, other important construction controls were lacking in this project. There was no architectural or engineering inspection of the project, inspection by the City of Boston was ineffective, and the control exercised by the Contractor and Subcontractors was inadequate.

1. Although the specifications prepared by the designers contain many references to "on approval of the architect", "shall be submitted to the architect for approval", etc., no architect or engineer was retained to see that the design was implemented.

   Mr. Kanieff, the general superintendent for the Structural Subcontractor, testified that he had never seen or spoken to a representative of either the Project Architect or Structural Engineer. Mr. Fontaine, the only field representative for the General Contractor, testified that he had never seen a representative of either the Project Architect or Structural Engineer at the job site. In addition, they indicated that no other registered engineer or architect provided inspection.

2. The Building Inspector for the City of Boston testified that he made 35 site inspections during construction. Mr. Fontaine testified that he had seen the Building Inspector only two or three times in February or March 1970. Mr. Kanieff testified that he had seen the Inspector perhaps 10 times during the first months of construction.

More important than the number of City inspections was their quality. Although the Inspector expressly admitted that one of his purposes for being on the job was to see that the Affidavit Engineer was present (Section 111 (h) of the Building Code requires the Building Commissioner to see that the Affidavit Engineer supervises construction), his 35 reports fail to note that neither the Affidavit Engineer nor the Licensed Builder of record was ever at the job site. Also, he testified that he never checked concreting, or curing, or any other structural aspect of the construction.

3. In the absence of professional inspection, a responsible general contractor will substitute his own forms of construction control. In this case, the General Contractor employed only one man at the site. This man was not a licensed builder. It is clear from his testimony, as well as that of others, that he did not direct in any detail the work of the Subcontractors and did not supervise or inspect their work. In essence, the construction planning exercised on this project was such as was mutually arranged by the Subcontractors.

5.

CONCLUSIONS AND RECOMMENDATIONS

The community was shocked that a tragedy of the magnitude that occurred on January 25 could happen. Indeed the formation of our Commission and our charge reflects this concern.

Over the years the building industry, like other aspects of society, has become more complex. With the development of ever more sophisticated technology and of complex financing and contractural arrangements, the laws and regulations controlling construction have been expanded in an effort to protect health and safety and the public interest. As a consequence of this increased complexity, the realization of any major building project requires the interactive cooperation of many individuals, organizations, and agencies. It must be recognized that no set of laws, regulations, or procedures can, by themselves, prevent building failures. As a case in point, the failure at 2000 Commonwealth Avenue WOULD NOT HAVE OCCURRED had the existing laws, regulations, and accepted procedures been understood and mutually respected by the several parties involved. We believe that this fact contains the most important lesson to be learned from the tragedy.

The loss of life in this tragedy is particularly distressing. We wish to express a sense of dismay that the project could have proceeded through so many successive phases without anyone recognizing and making appropriate

remonstration against the irregularities and deficiencies which took place. The public interest cannot tolerate a repetition of this series of events.

Recognizing our principal conclusion, we, nevertheless, feel that improvements in law, procedures, and technology can be made which will diminish the risk of future building failures of this kind. Particularly we believe that improvements should be made:

1. In the methods of assigning responsibility and of ensuring competence for the design and construction, including inspection, of major buildings.

2. In the organization and in the staff competence of the Building Department.

3. In building codes relative to flat slab reinforced concrete constructions, so that a local failure at one point of support is less likely to propagate a general collapse.

Additionally, it would be possible to suggest improvements pertaining to almost every one of the irregularities and deficiencies listed in Section 4. Instead, we commend to the Mayor only four recommendations. The first three pertain in turn to the three areas for improvement noted above, and can be accomplished by the City. The fourth recommendation pertains to the first area for improvement, but involves actions which can come only from the state legislature.

Recommendation 1

In order to make owners fully aware of their basic responsibility in the development of a building project and in order to ensure that owners engage adequate

and competent professional services for all aspects of design and construction, we urge revisions in Section 110, Application for Permit, and in Section 111, Permits, in the Building Code of the City of Boston.

Section 110 now allows the Building Commissioner to accept, without detailed review, an application for a building permit which is accompanied by plans and computations and an affidavit filed therewith by a competent architect or engineer to the effect that said plans and computations are in accordance with the requirements of the Building Code. When this procedure is followed, Section 111 requires the Building Commissioner to see that the affidavit architect or engineer supervise construction, be responsible for its conformity with the approved plans, and forthwith upon completion of construction, file with the Building Commissioner a second written affidavit that the work has been done in conformity with the approved plans and the structural provisions of the Code.

In practice, the affidavit form is signed by the structural engineer who can in fact certify to the correctness only of his structural documents. No one certifies as to the conformity of the architectural or mechanical documents with the building code. There is no mention on the first affidavit form that the signatory must supervise and at completion file a second affidavit.

The affidavit procedure can be made more effective if its provisions and the affidavit forms are revised to place first responsibility on the owner and to require the signature of the licensed individuals who were responsible for the preparation of each aspect of the documents and for the inspection of the construction.

The owner should be made fully aware of his duty to conform with the Boston Building Code and to provide for the safety of all persons. The owner controls

a project through the quality of the individuals and organizations he retains for its design and construction and is in the best position to ensure that the necessary quality control - so lacking in the development and construction at 2000 Commonwealth Avenue - is exercised. To meet his reponsibilities he must provide professional services both for design and inspection to be performed by individuals licensed to practice in the Commonwealth.

Section 110 of the Building Code should be revised to define certain building project limits, above which use of the affidavit provisions would be mandatory to issuance of a building permit. Section 111 does not define "supervision" or indicate the nature or extent of the supervision services which will be required. The word "supervision" is no longer used in agreements which define architectural and engineering services. To conform with current usage the word "inspection" should be substituted.

The extent and regularity of inspection services which are reasonably necessary vary greatly with the scope of each project, the type of construction, the materials used and the type of mechanical installations. To facilitate implementation of a more precise affidavit procedure comprehensive requirements for inspection of construction should be established either by code amendment or by Commissioner's bulletin which will specify the nature and degree of inspection to be required for each type of construction, construction material, and mechanical installation. These requirements must recognize that a lesser degree of inspection is adequate on straightforward conventional projects and installations of limited scope than is appropriate on large or complex projects.

Any major building involves architectural, structural, mechanical, and electrical aspects. The plans for each aspect are normally prepared under the

direction of different individuals. Whether or not the affidavit procedure is used, the Commissioner should require that all plans, specifications, and computations submitted as part of a building permit application carry the initialed Massachusetts registration seal of the individual responsible for their preparation unless the project is of small scope as defined in Massachusetts General Laws, Chapter 143, Section 45A.

Whenever a building permit is to be issued in accord with the affidavit procedure we recommend the filing of two affidavit forms:

First Affidavit

The first affidavit would be filed with the plans and computations and become part of the building permit application.

This affidavit would have two objectives. First, the licensed Architect and Engineers engaged by the owner would certify that their documents conformed with the Building Code of the City of Boston. Second, the Owner would obligate himself to retain for construction a signatory Licensed Builder and such signatory licensed professionals as may be required to satisfy the inspection provisions of the Code. If for any reason the connection of the Licensed Builder or any professional inspector is terminated the Owner would be obligated to stop construction unless the Commissioner had approved a substitution on the affidavit form.

The Licensed Builder would agree to direct the construction in accord with the Code. The Architect and Engineers engaged for inspection would agree to render the appropriate inspection services, the code requirements for which would be referenced or attached to the affidavit. The Licensed Builder,

Architect, and Engineers would agree to sign forthwith on completion a second affidavit and, if their responsibilities in connection with the work were for any reason suspended or terminated to file with the Commissioner a sign-off statement. The Owner's signature would be notarized, the Licensed Builder's number would be entered and the Architect and Engineers' signatures would be accompanied by their initialed Massachusetts registration seals.

Second Affidavit

Filing of the second affidavit would be required immediately on completion of construction and would be prerequisite to issuance of a certificate of occupancy.

The Licensed Builder would certify that all construction had been conducted in accord with all construction provisions of the Code and in agreement with the approved plans or subsequent changes which would have to be certified by an appropriate licensed professional as to their conformity with the Code.

The Architect and Engineers would each certify that he had provided the inspection required by the first affidavit, that the construction conformed both with the inspection requirements of the Code and substantially with the approved plans or subsequent approved modifications. The Licensed Builder's number would be entered and initialed Massachusetts registration seals would be affixed.

We believe that these changes will bring to the owner increased awareness of his responsibility and will lead to improved quality in design and construction.

Recommendation 2

In order that the Building Department may be better able to fulfill its role in the building process, we urge an upgrading of the administrative procedures by which it attempts to accomplish its objectives and of the technical competence of the staff which must implement these procedures.

During the last decade Boston has experienced a building boom. Not only has there been a tremendous increase in the number and size of building projects, but the complexities of development methods, design types, and construction methods have increased significantly. In direct proportion to this boom, the demands upon the Building Department have increased.

In the last few years, the Building Commissioner has implemented a new building code and a number of more rational administrative policies and procedures. Since the tragedy at 2000 Commonwealth Avenue he has instituted a series of inspection forms which constitute a check list for the city building inspectors and provide a record of the conditions found at each inspection.

Commendable as these past and current improvements may be, much more must be done if the Building Department is to function effectively. We recommend that the City undertake a complete review of the entire operating policy of the Department and develop, with the Commissioner, a total administrative system commensurate with current and future needs.

In our principal conclusion we noted that no set of regulations or procedures are, themselves, sufficient. No administrative system, however good, can be effective if those on the operative staff are not capable of discharging their responsibilities. To protect properly the interests of the citizens of Boston,

the staff of the Building Department must have sufficient competence to function effectively in the building process.

Among the current staff of the Building Department, which numbers about 130 employees, the Commissioner and Mr. Justin O'Farrell are the only registered professional engineers. The staff does not include a registered architect. Only three employees have a degree from a 4-year college or university. Two of these are legal assistants and the other is the Commissioner, who has a degree in civil engineering.

These facts must be considered in relation to the technical nature of the building industry. One might liken the current situation in the Building Department to a situation wherein one attempts to operate a Law Department without any lawyers. We believe that the technical competence of the Building Department must be upgraded.

Recommendation 3

In order to assist the engineering profession in its continuing development of code procedures for the safe design of reinforced concrete flat slab constructions, we urge that the technical aspects of our report be made generally available.

We believe that such release would bring to bear on the construction industry the lessons which we have learned from this tragedy and, in consequence, serve to upgrade the standards of the industry. Even more importantly, however, we believe that the propagating collapse experienced at 2000 Commonwealth Avenue deserves the attention of those individuals, agencies, and organizations who have the responsibility to develop codes and standards of practice for the design of reinforced concrete flat slab buildings.

Hundreds of reinforced concrete flat slab buildings have been constructed and the vast majority have performed satisfactorily. In spite of this fact we believe that structural engineers, in both research and practice, will wish to examine the particular characteristics of this structure and the nature of the progressive collapse which occurred.

A concern for progressive collapse of buildings was raised in England in 1968 as a consequence of the failure of a precast concrete apartment building in a development known as Ronan Point. The precast structure of this apartment, consisting of floor and wall panels assembled at the site, was wholly different from the cast-in-place structure at 2000 Commonwealth Avenue. The initial cause of failure was a gas explosion and the nature of the progressive collapse quite different. However, an investigating Commission, similar to our own, recommended the development of "code" requirements which would prevent progressive collapse in such precast concrete apartment buildings. The authorities in England subsequently developed requirements and today designers must demonstrate that removal of any one element of a structure will not lead to a progressive collapse.

Many research programs have been conducted relating to the shear strength of a flat slab around a single column, but it appears that more study should be given to the conditions of a complete floor slab in the event of shear failure around one column. At 2000 Commonwealth Avenue a shear failure around one column propagated laterally to include an entire half of the structure at both the main roof and later at the 16th floor.

The principal parameters which influence this type of behavior are loading, span lengths, and quantity and distribution of steel reinforcement. Future

research should be able to determine the limiting conditions under which such lateral propagation would be expected (or precluded). It should be possible to arrive at design procedures which would preclude such failures.

Our report has been organized to facilitate this third recommendation. The technical content is contained in Part I, Sections 1C, 2, 3, 4B through 4D, and Part II.

Recommendation 4

In order to establish an effective relationship between state requirements and needs of the Boston Building Department, we recommend state legislation to amend existing statutes regarding registration and practice of architects and engineers in Massachusetts.

A. Massachusetts General Laws, Chapter 112, Sections 81D-81T, relative to registration of professional engineers and of land surveyors, should be amended to require that professional engineers be registered specifically as structural, mechanical, electrical, and/or sanitary engineers and that their seals of registration carry the appropriate designation.

B. Massachusetts General Laws, Chapter 112, Sections 81D-81T, relative to registration of professional engineers and of land surveyors, and Chapter 112, Section 60 A to 60 L, relative to registration of architects, both should be amended to make it clear that preparation of plans for structures to be erected in Massachusetts constitutes practice in Massachusetts no matter where such preparation is carried out.

C. Massachusetts General Laws, Chapter 143, Section 54A, pertaining to the approval of plans by building departments and other authorities, should be amended to require:

1. That all plans carry the initialed seal of registration of the architect or engineer responsible for their preparation unless the plans pertain to small projects currently exempted from the provisions of this section.

2. That the architectural plans carry the initialed seal of a registered architect, unless the building department rules that the architectural work is minor and can be considered incidental to other work.

3. That the structural, mechanical, electrical, and sanitary plans carry the initialed seal of a professional engineer licensed in the appropriate specialty, unless the building department rules that a particular aspect of the work is minor and can be considered incidental to other work.

These changes key directly to our suggested procedure for implementing Recommendation 1.

APPENDIX I.1

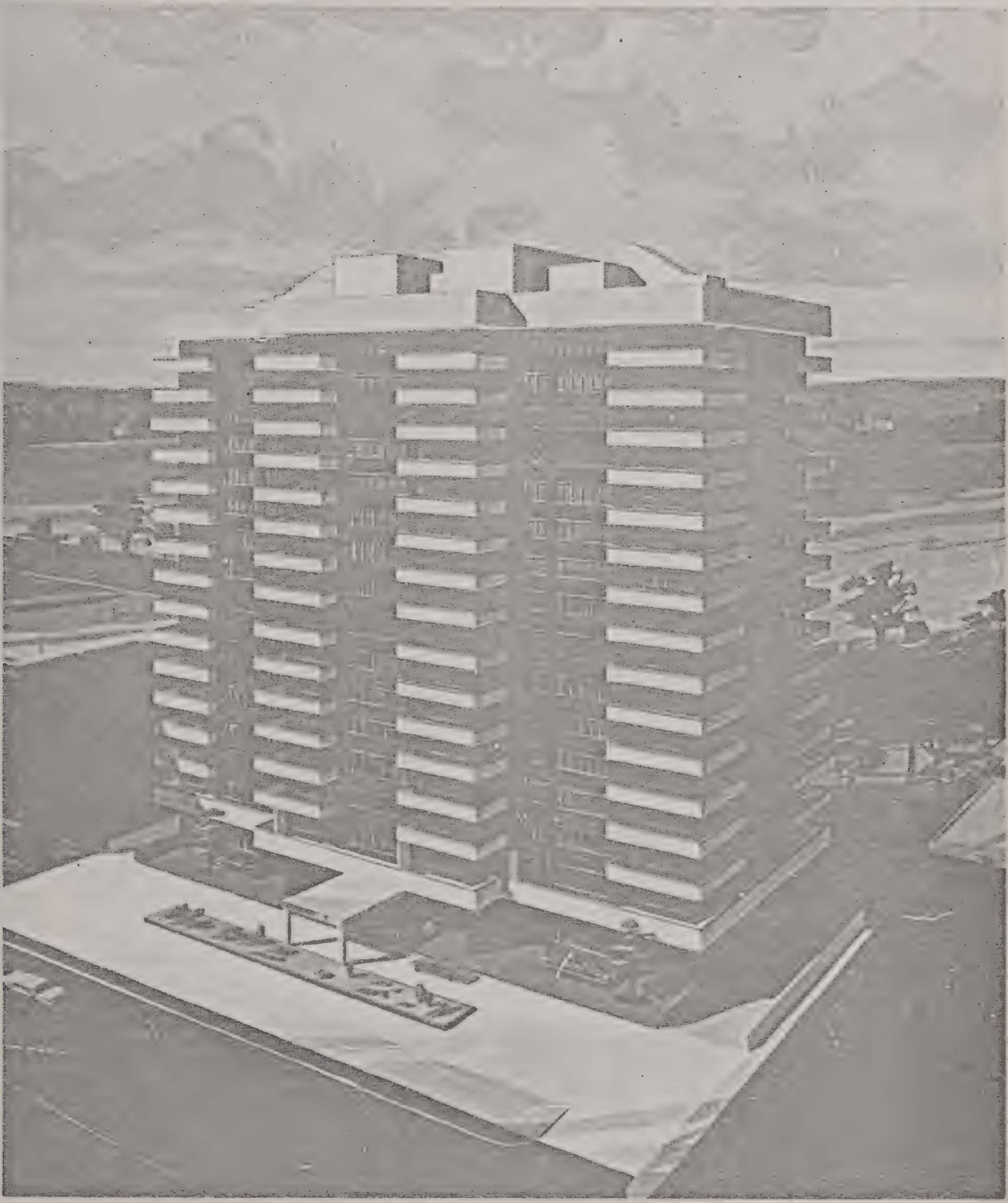
FIGURES

1. Architectural Rendering Of The Designed Building
2. Photograph Of Constructed Building Taken Two Hours Before The Collapse
3. Schematic Building Plan
4. Typical Floor Plan
5. Main Roof And Mechanical Floor Plan
6. Photographs Taken After Collapse Of The Roof Slab But Before The General Collapse
7. Photograph Of The General Collapse In Progress
8. Photographs Showing The Aftermath Of The Collapse
9. Eyewitness Plan: Roof
10. Eyewitness Plan: 16th Floor
11. The Extent Of The Collapse

TABLES

1. Test Results For Roof Slab Concrete Placed on December 9, 1970.
2. Test Results For Roof Slab Concrete Placed on December 3, 1970.

FIGURE 1. ARCHITECTURAL RENDERING OF THE DESIGNED BUILDING



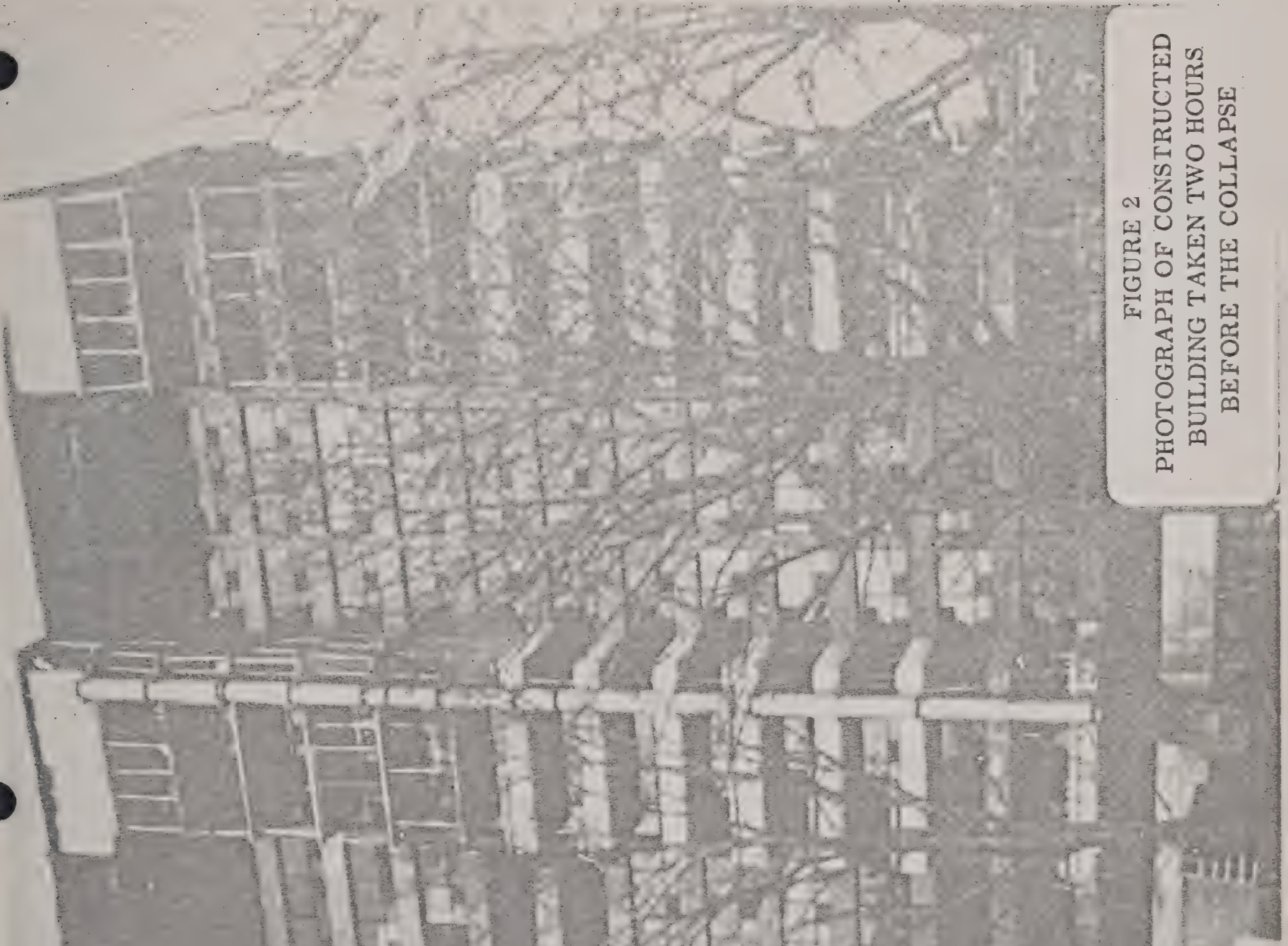


FIGURE 2
PHOTOGRAPH OF CONSTRUCTED BUILDING TAKEN TWO HOURS BEFORE THE COLLAPSE

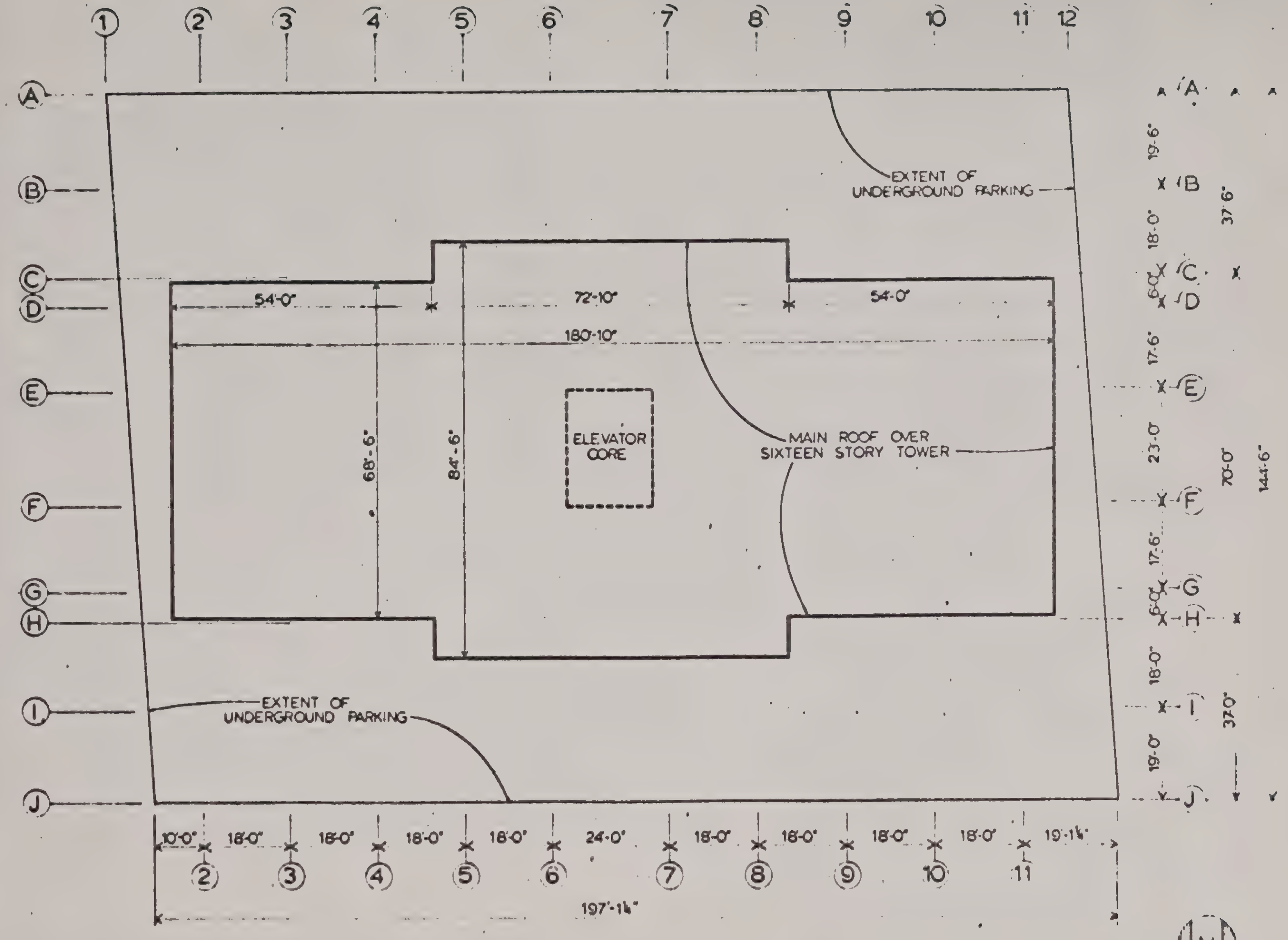


FIGURE 3 SCHEMATIC BUILDING PLAN

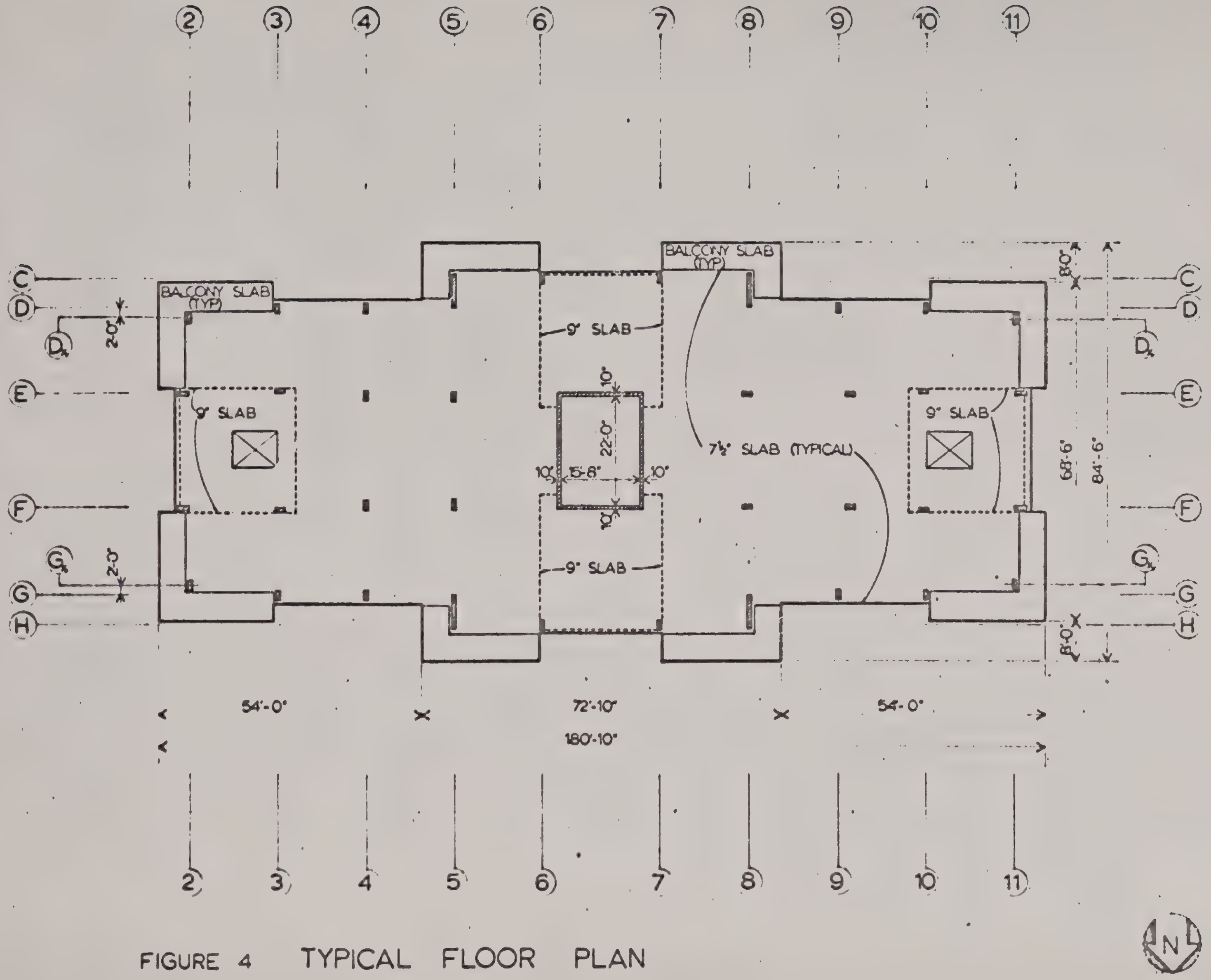


FIGURE 4 TYPICAL FLOOR PLAN

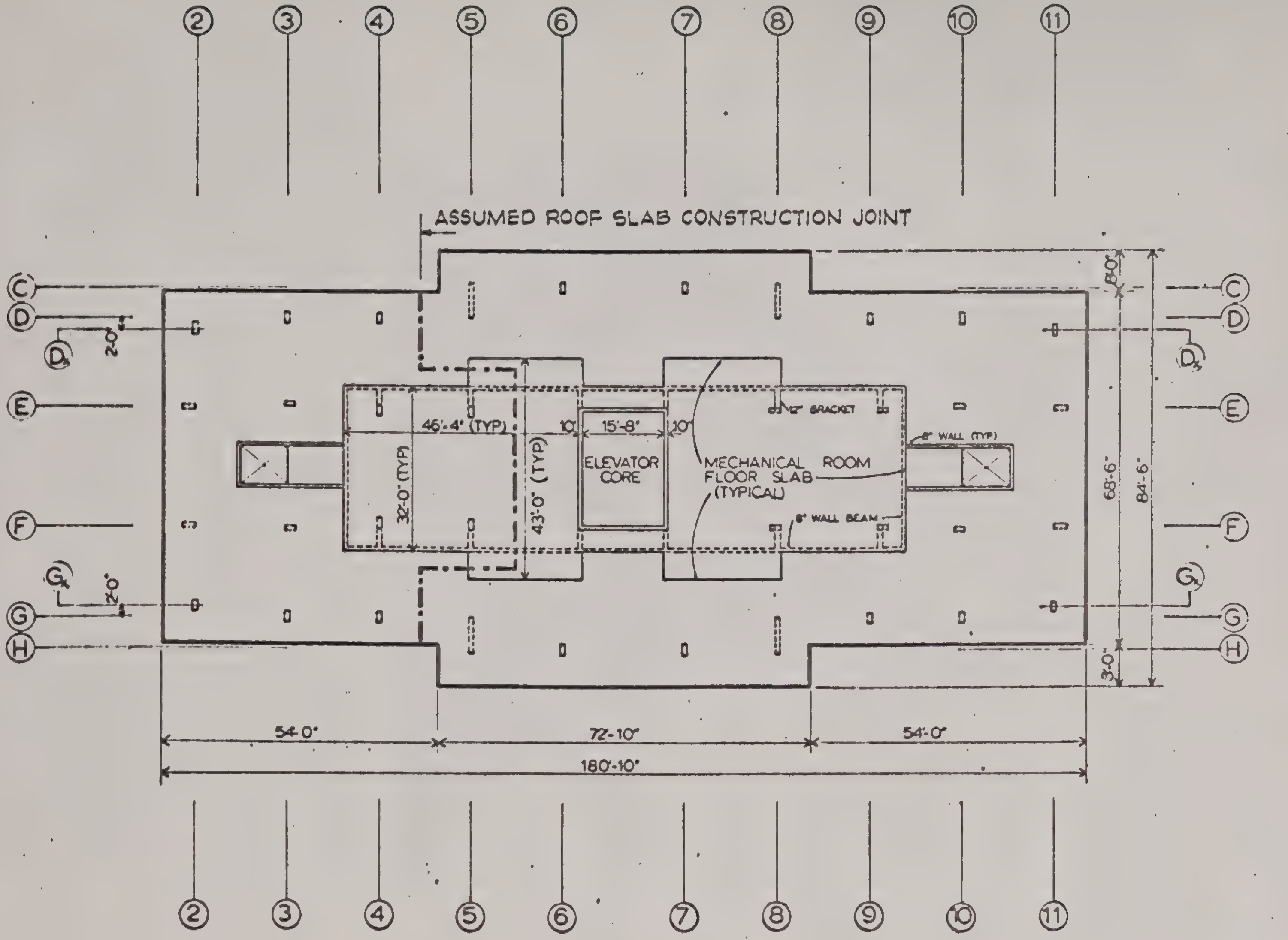
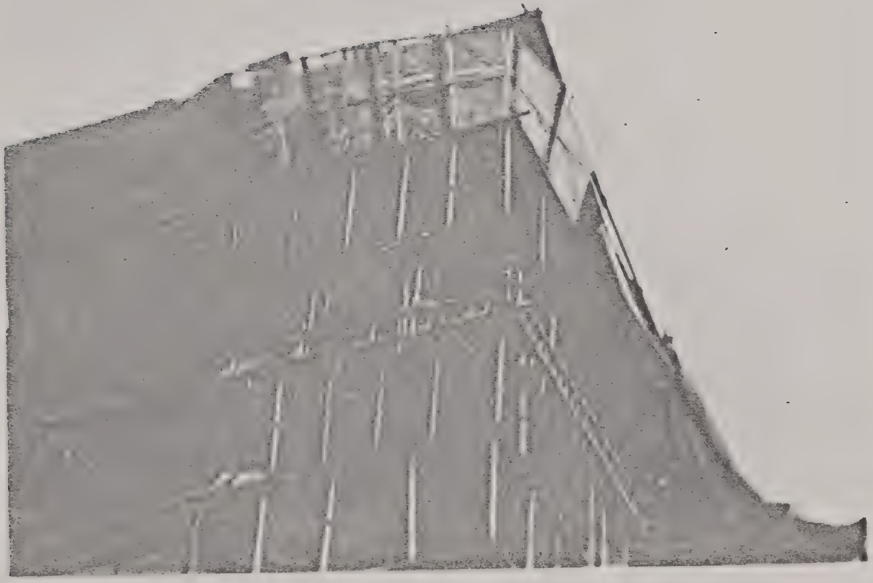


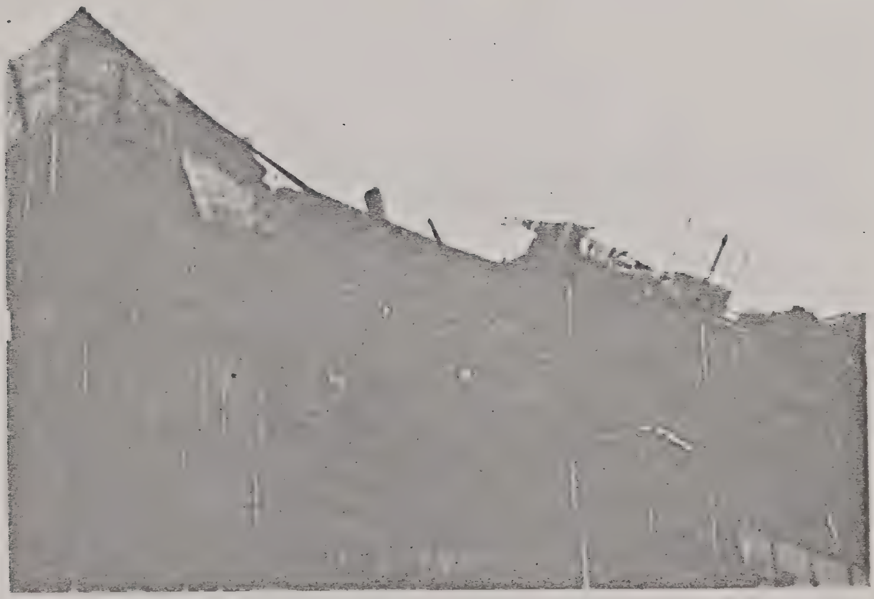
FIGURE 5 MAIN ROOF AND MECHANICAL FLOOR PLAN

FIGURE 6. PHOTOGRAPHS TAKEN AFTER COLLAPSE OF THE ROOF SLAB BUT BEFORE THE GENERAL COLLAPSE



(a) Northeast Corner Facing West South West

FIGURE 6. Continued



(b) Northeast Corner Facing Southwest

FIGURE 6. Continued

(c) North Side Facing South

FIGURE 7. PHOTOGRAPH OF THE GENERAL COLLAPSE IN PROGRESS

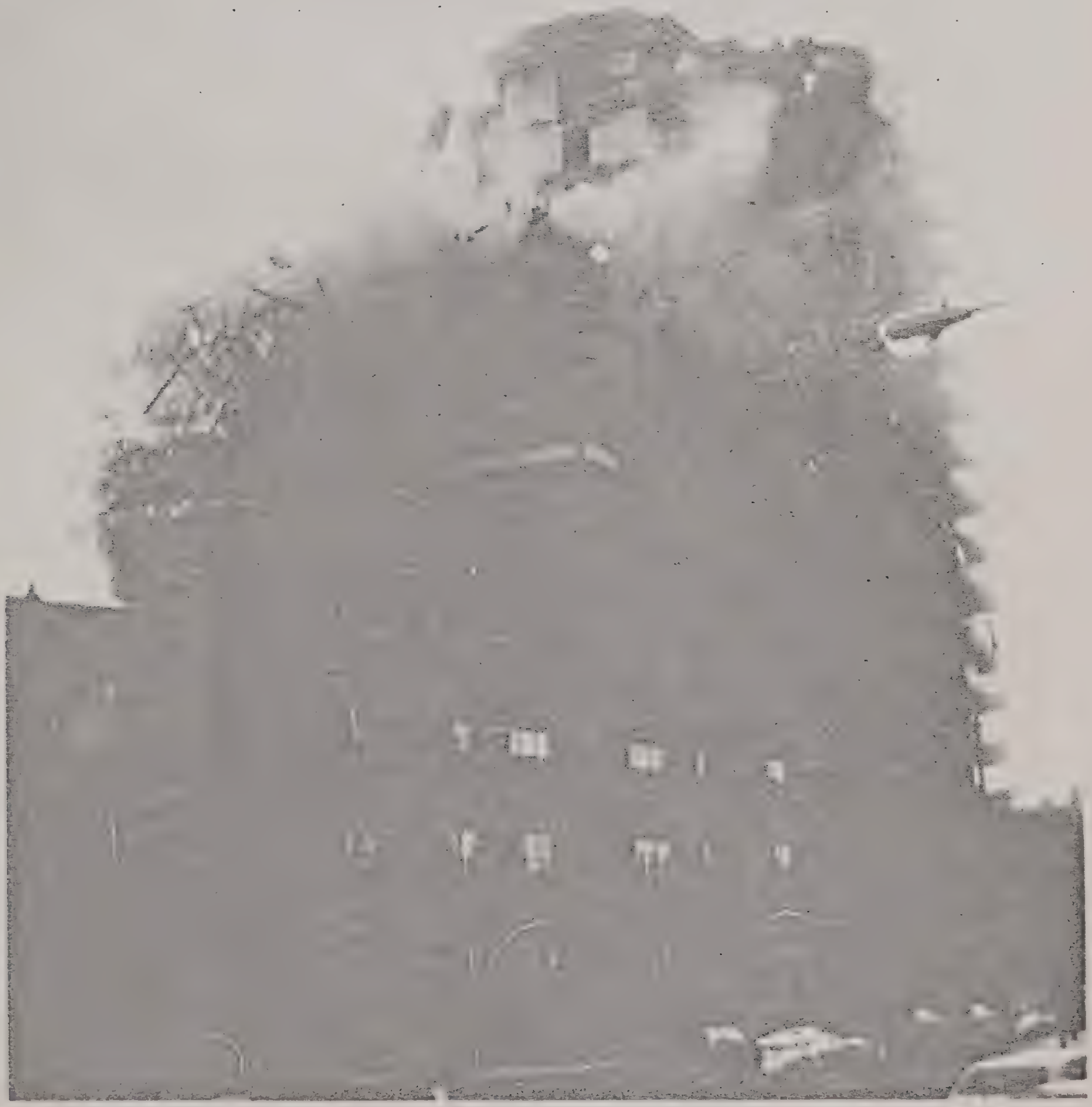
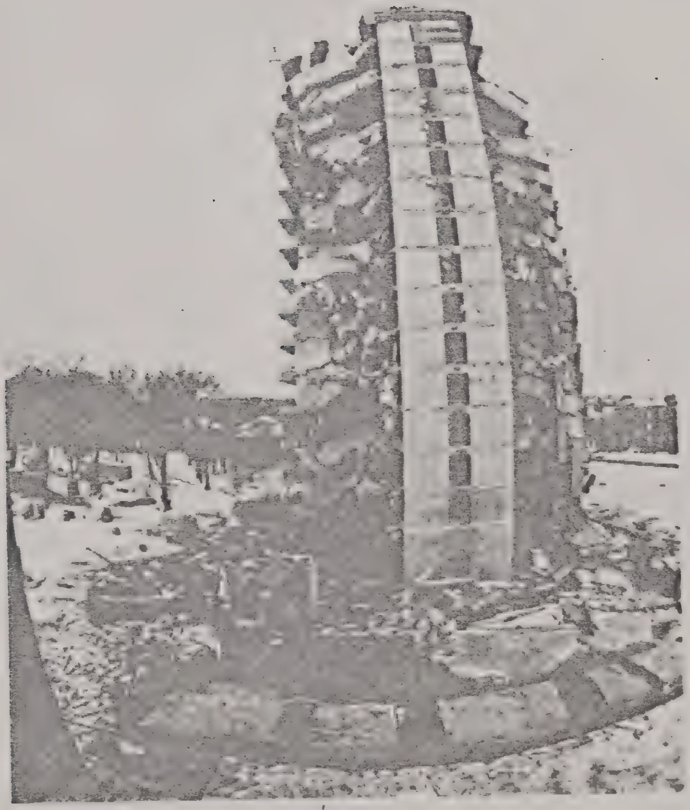
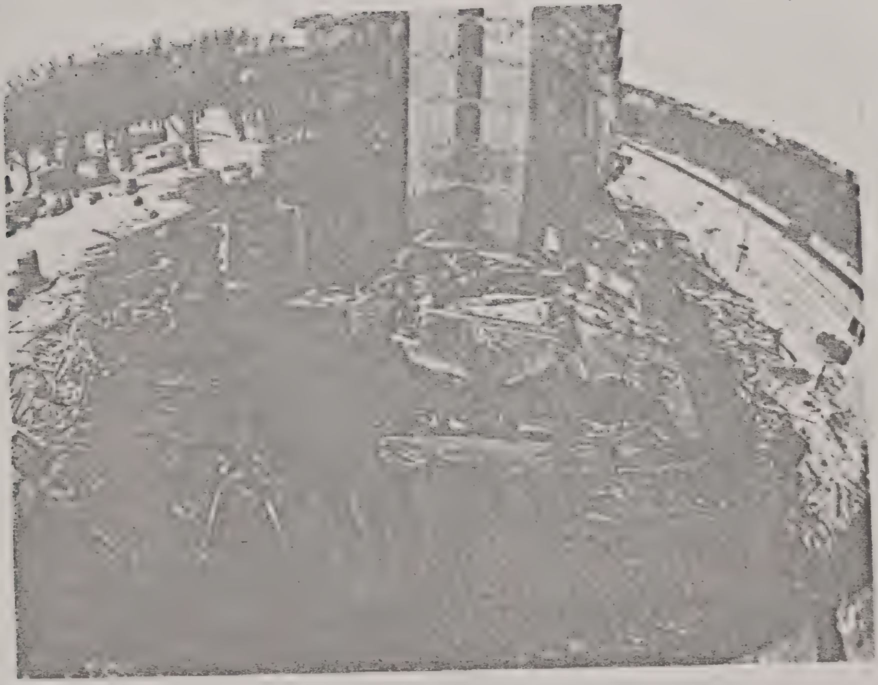


FIGURE 8. PHOTOGRAPHS SHOWING THE AFTERMATH OF THE COLLAPSE



(a) Taken With A "Fisheye" Lens Facing West

FIGURE 8. Continued



(b) Taken With A "Fisheye" Lens Facing West

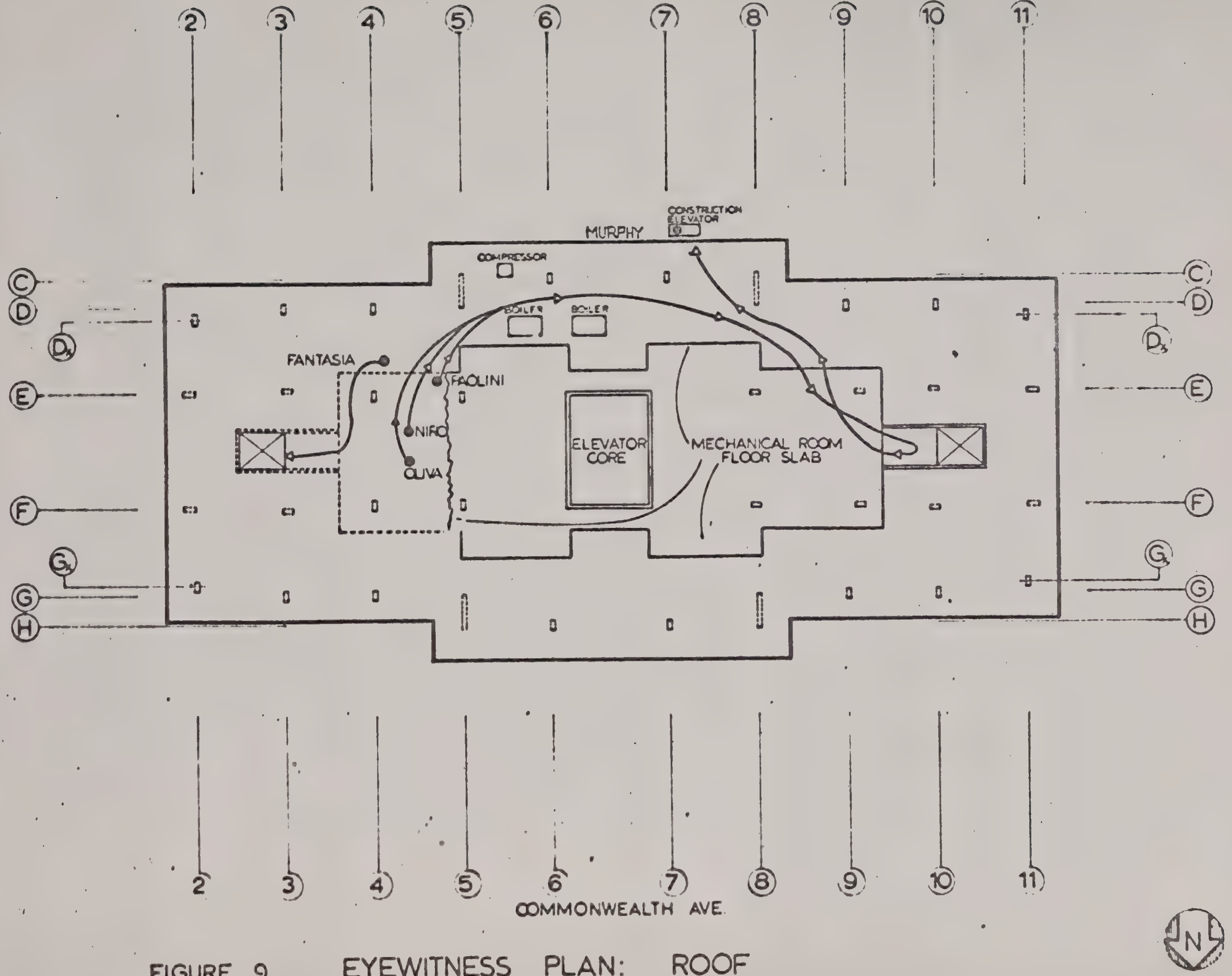


FIGURE 9 EYEWITNESS PLAN: ROOF

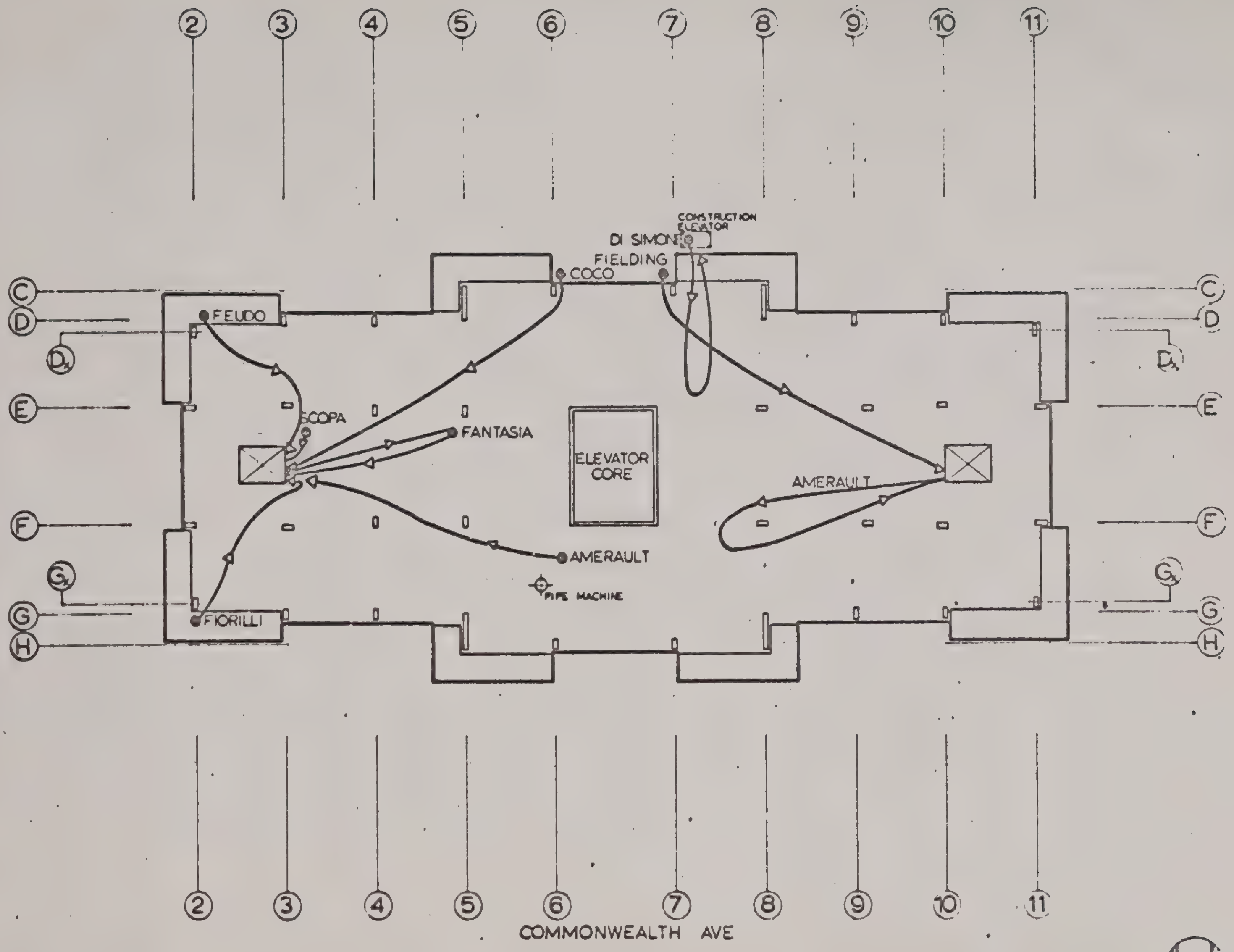


FIGURE 10 EYEWITNESS PLAN: 16TH FLOOR

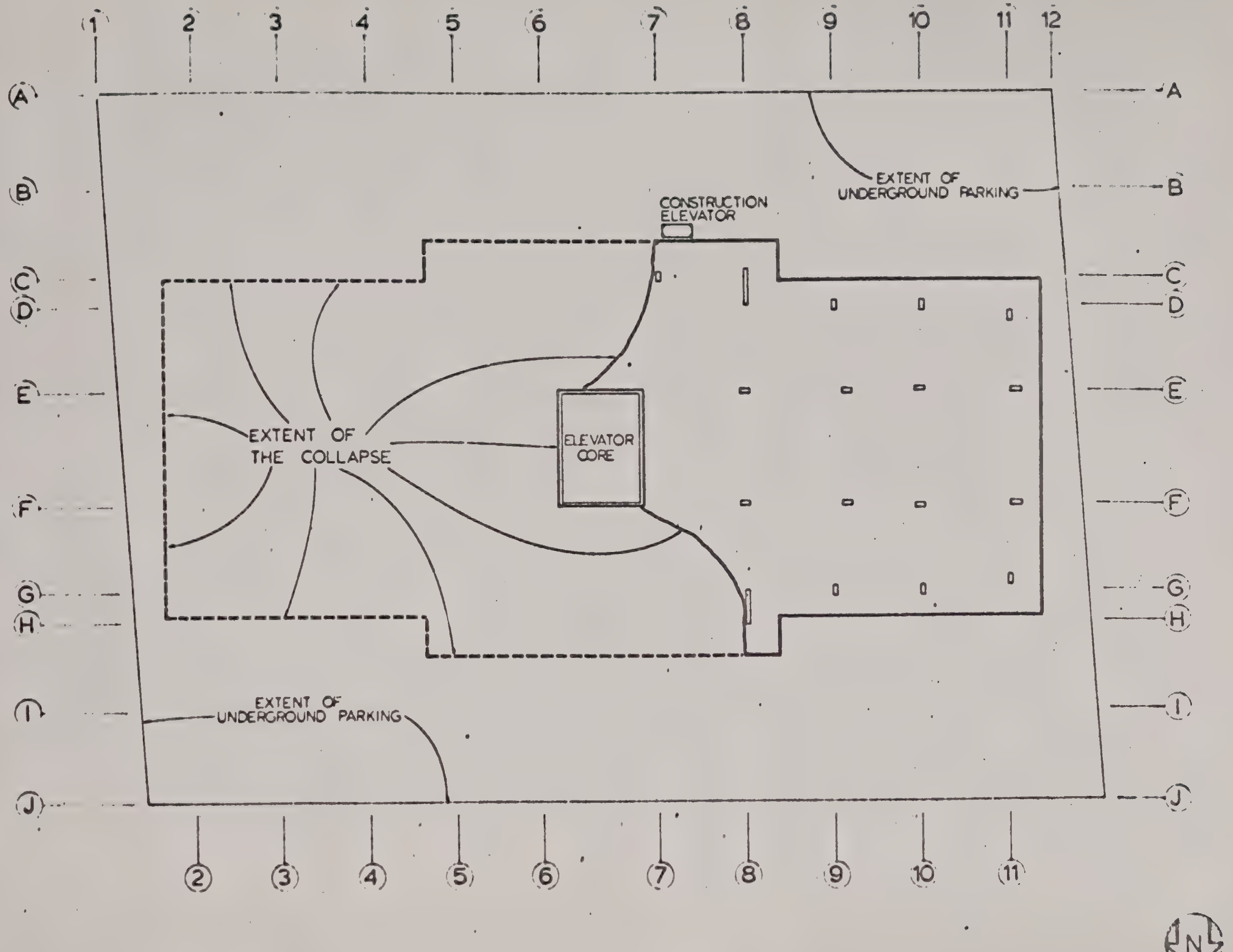


FIGURE 11 EXTENT OF THE COLLAPSE

TABLE 1

TEST RESULTS FOR ROOF SLAB CONCRETE PLACED ON DECEMBER 9, 1970

Location	Core Diameter (inches)	Date Sampled	Date Tested	Age at Test (days)	Density (lbs./cu.ft.)	Actual f'_c (psi)	Comments	Cement Content (lbs./cu.yd.)
W 9	4	2/6/71	2/9/71	61	146.9	2100	Sound Appearance	
(D$\frac{3}{4}$-2)	4	2/6/71	2/8/71	61	146.4	1920	Sound Appearance	
	4	2/6/71	2/8/71	61	146.9	2090	Sound Appearance	566
	4	3/23/71	3/25/71	106	146.6	2760	Sound Appearance	490
	4	3/23/71	3/31/71	112	146.9	2660	Sound Appearance, Core Steam cured for 5 hours at pressure of 150 psi and temperature of 300°F	
W 11	4	3/23/71	3/25/71	106	147.6	2310	Tested core cracked from top to bottom	492
(D$\frac{1}{4}$-2)	4	3/23/71	3/25/71	106	N.A.	2970	Sound Appearance, Tested core contained two reinforcing bars	

TABLE 1 (continued)

TEST RESULTS FOR ROOF SLAB CONCRETE PLACED ON DECEMBER 9, 1970

Location	Core Diameter (inches)	Date Sampled	Date Tested	Age at Test (days)	Density (lbs./cu.ft.)	Actual f'_c (psi)	Comments	Cement Content (lbs./cu.yd.)
H 20	4	2/7/71	2/9/71	62	149.4	1370	Sound Appearance	488
(D-2¾)	4	2/7/71	2/9/71	62	147.3	1570	Sound Appearance	
	4	2/7/71	2/9/71	62	146.3	1330	Tested core cracked through middle	
	4	3/23/71	3/25/71	106	146.8	2360	Sound Appearance	530
	4	3/23/71	3/25/71	106	145.8	1750	Tested core badly cracked up 3" from base	
	4	3/23/71	3/25/71	106	146.8	2050	Tested core cracked completely across	

TABLE 2

TEST RESULTS FOR ROOF SLAB CONCRETE PLACED ON DECEMBER 3, 1970

Location	Core Diamter (inches)	Date Sampled	Date Tested	Age at Test (days)	Density (lbs./cu.ft.)	Actual f'_c (psi)	Comments	Cement Content (lbs./cu.yd.)
D-10$\frac{1}{2}$	4	3/13/71	3/15/71	102	N.A.	2710	Sound Appearance, Tested core includes end of #5 reinforcing bar	537
F$\frac{3}{4}$-10$\frac{3}{4}$	4	3/18/71	3/20/71	107	146.5	2580	Sound Appearance	493
E$\frac{1}{2}$-9$\frac{3}{4}$	4	3/18/71	3/20/71	107	146.5	2050	Sound Appearance	512
F$\frac{1}{2}$-9$\frac{1}{2}$	4	3/13/71	3/15/71	102	145.4	2240	Sound Appearance	514
E$\frac{1}{2}$-7$\frac{3}{4}$	4	3/13/71	3/15/71	102	144.6	2800	Sound Appearance	556
D$\frac{1}{2}$-7$\frac{1}{4}$	4	3/13/71	3/15/71	102	144.6	3010	Sound Appearance	536

TABLE 2 (continued)

TEST RESULTS FOR ROOF SLAB CONCRETE PLACED ON DECEMBER 3, 1970

Location	Core Diameter (inches)	Date Sampled	Date Tested	Age at Test (days)	Density (lbs./cu.ft.)	Actual f'_c (psi)	Comments	Cement Content (lbs./cu.yd.)
Π 15	2	2/5/71	2/8/71	67	148.9	2390	Sound Appearance	
(D-6¼)	2	2/6/71	2/8/71	67	148.2	2840	Sound Appearance	
	2	2/6/71	2/8/71	67	148.4	2390	Sound Appearance	
	4	2/6/71	2/8/71	67	145.9	1880	Tested core cracked from top to bottom	
	4	3/23/71	3/25/71	112	145.8	2300	Tested core had hair-line crack extending down 3"	

APPENDIX I. 2

LIST OF INDIVIDUALS INVITED TO TESTIFY

Not Appearing

Bartholomew W. Cosentino, one of the owners, 2000 Commonwealth Avenue
William L. Little, retired plans examiner, Boston Building Department
M. S. Yolles, structural engineer, Toronto, Canada

Appearing

2/10/71	Dante F. Montouri, professional engineer, Wayland, Massachusetts
	Pierre Fontaine, job superintendent, Toulon Construction, Inc.
	Boris Kanieff, general superintendent, Fountain Construction Co.
	Frank Di Simone, ironworker foreman, Fountain Construction Co.
	Anthony Paolini, labor foreman, Fountain Construction Co.
2/17/71	Matthew Lafferty, district building inspector, Boston Building Dept.
	Justin O'Farrell, supervising plans examiner, Boston Building Dept.
	Paul A. Scopa, masonry foreman, Pizzotti Brothers, Inc.
	Edward J. Amerault, steamfitter

2/24/71	Christopher Feudo, bricklayer
	Kenneth Fielding, bricklayer
	Albert Fiorilli, bricklayer
	Alfred Coco, bricklayer
	Phillip J. Burne, architect, Saugus, Massachusetts
	Richard R. Thuma, Jr., building commissioner
3/3/71	Antonio M. Fantasia, carpenter foreman, Fountain Construction Co.
	Angelo Sirico, carpenter foreman, Fountain Construction Co.
	Frederick T. Boyle,Jr., manager, Hub Testing Laboratories
	John H. McNamara, Jr., vice president, J.H. McNamara, Inc.
3/24/71	Joseph J. Lane, vice president, Northern Steel, Inc.
	Joseph Oliva, cement finisher
	Daniel Niro, cement finisher
4/14/71	Victor G. Dragone, notary public, Everett, Massachusetts
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THE BUILDING COLLAPSE AT
2000 COMMONWEALTH AVENUE
BOSTON MASSACHUSETTS

on

January 25, 1971

REPORT

OF

THE MAYOR'S INVESTIGATING COMMISSION

PART II

JUNE 1971

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1.

STRUCTURAL ANALYSIS OF THE FAILURE CONDITIONS

Analyses of both the east and west halves of the main roof slab and the east half of the 16th floor slab were made. These analyses consisted of two basic parts: (1) a determination of the gross internal forces (i.e., axial forces, shear forces, and bending moments) induced in the slab and its supporting columns and walls according to the assumptions of elasticity and (2) a determination of the capacity of a particular portion of the slab to resist the previously determined forces.

There are several analytical procedures which can be used to calculate gross internal forces in the roof slab. All of these procedures account for static equilibrium with the applied loads and they vary only in the degree to which they simulate the continuity of the actual structure. The so-called finite element method of analysis allows one to most closely simulate the actual conditions in the roof slab of the structure at 2000 Commonwealth Avenue. Although the finite element method of analysis normally is not used in the design of such a structure, it is considered to be the most accurate practical method of analysis which is currently available and it was used in this investigation.

The capacity determinations were based on procedures and formulae drawn from published analytical and experimental research.

A. EAST SIDE OF MAIN ROOF

1. Finite Element Analyses

Seven finite element analyses were made for the east half of the main roof. Six

of the analyses were based on the grid shown in Figure 1.1. The accuracy of a finite element analysis which incorporates rectangular elements is generally thought to be improved as the number of elements in the model is increased and as the elements themselves become more square. Because the grid in Figure 1.1 is relatively coarse and because it incorporates several elongated elements, one analysis using a finer grid was made. It was based on the grid shown in Figure 1.2.

Grid Shown in Figure 1.1

In Figure 1.1 the roof slab, from the east edge to line 8, is divided into 176 rectangular elements. In general the element thickness is taken as 7 1/2 inches but in the three areas shown by crossed lines the slab thickness is taken as 9 inches. Of course, the roof does not terminate in line 8, but the results determined for areas east of line 7 will be reasonable accurate provided that one introduces sensible boundary conditions into the model along line 8.

The finite element model includes supporting elements, each of which is assumed to be rigidly attached to the roof slab, to extend down 9 feet 6 inches (to the 16th floor), and to be fixed at its base. In the model, columns 1 through 12, 15, 16, 19, 20, 31, and 32 represent actual columns in the real structure. Columns C5 and H5 in the actual structure are each represented in the model by a rigid frame consisting of two columns and a strong beam. For example, column C5 is represented by column 17, beam 34, and column 18. In five of the six analyses the elevator core walls are represented by the ten columns numbered 21 through 30. In one analysis the accuracy of this last representation was tested by representing the elevator core walls as a group of finite elements rather than as a group of columns.

Careful comparison of the grid layout shown in Figure 1.1 with the contract documents will reveal some discrepancies. Columns 5, 8 through 14, and 17 through 32 are correctly located on the finite element grid, but the positions of columns 1 through 4, 6, 7, 15, and 16 are not exactly reproduced. The Y coordinate of column 16 (column E5), for example, should be 10.917 feet instead of 11 feet. Other minor deviations exist in both the extent and thickness of the roof slab itself. For example, the Y coordinate of joints 12, 24, 36, 48, 60, and 72 should be 34.25 feet instead of 35 feet and the extent of the slab portions which were thickened to 9 inches is not precisely reproduced in the finite element grid. By using many more elements one could develop a grid which would precisely represent the actual conditions. Such an effort is not justified because it would result in large computer programming and computer time costs for small increases in the precision of the results.

Each of the analyses had a particular purpose:

Run 1

> Assuming that there was no shoring, determine the bending moments, shears, and thrusts throughout the roof slab and its supporting columns for a loading consisting of the weight of the roof slab (94 pounds per square foot for 7 1/2 inch thick portions and 112 pounds per square foot for 9 inch thick portions); the formwork for the mechanical room floor (18 pounds per square foot); the freshly placed mechanical room floor slab (75 pounds per square foot for 6 inch thick portions and 112 pounds per square foot for 9 inch thick portions), wall beams, and brackets (assuming placement from line 8 to line 5); two boilers (6 kips each); one compressor (2 kips); and some miscellaneous construction materials such as reinforcing bars (1 kip). These loadings are documented in Part II, Section 5.

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Run 2

Evidence indicates that there was a series of shores under the overhanging portion of the east half of the roof slab along the entire perimeter. To determine whether such shores could carry any load which is placed inside the exterior columns, Run 1 was repeated except that the dead weight of the overhanging roof slab elements was removed (e.g., elements 1 to 11, 22, 32, 43, etc.). In Run 2 it was also assumed that placement of concrete in the mechanical floor had progressed three feet east of line 5.

Run 2A

To check on the propriety of replacing the elevator core walls by a series of ten columns, this Run duplicates Run 2 except that columns 21 through 30 are replaced by ten (wall) elements, and 7 pounds per square foot are added for reinforcing steel in the yet unplaced mechanical room floor.

Run 3

Run 1 (and Run 2) indicated the presence of large slab bending moments perpendicular to the east, south, and north faces of the elevator core. These bending moments could not have been resisted elastically by the designed slab, and accordingly, yielding must have occurred at these locations. This Run is the same as Run 1, but assumes free rotation of the roof slab perpendicular to the east face, southeast face, and northeast face of the elevator core. (It also assumes that placement of the mechanical room floor and wall beams had progressed three feet east of line 5, that the reinforcing steel for the yet unplaced mechanical room

floor weighed 7 pounds per square foot, and that each boiler weighed 8 kips instead of 6 kips.)

Run 4

Eyewitness accounts (corroborated by analysis of available shear capacity to resist the forces determined in Runs 1, 2, and 3) indicated a slab shear failure around column 16 (column E5). This Run is the same as Run 3 except that column 16 is removed.

Run 5

As expected the results of Run 4 indicate a lateral transfer of loads and forces away from column 16. To give an indication of the continuation of such lateral transfer, this Run removes column 15 (column F5).

Grid Shown in Figure 1.2

The grid shown in Figure 1.2 is for only the southeast quarter of the roof slab. It contains the same grid lines as those in Figure 1.1, but further subdivides several of the elements. Symmetry boundary conditions are assumed along the edges at lines E 1/2 and 6 1/2. The elevator core walls down to the 16th floor are represented by two levels of finite elements.

The one analysis based upon the fine grid is identified as Run 2B. It incorporates the same loading conditions as the previously described Run 2A.

2. Shear Stresses in Flat Slabs Near Columns

References:

1. Hanson, N. W., and Hanson, J. M., "Shear and Moment Transfer Between Concrete Slabs and Columns," Journal PCA Research and Development Laboratory, V. 10, No. 1, Jan. 1968, pp. 2-16.

2. Moe, J., "Shearing Strength of Reinforced Concrete Slabs and Footings Under Concentrated Loads," Res. & Dev. Bulletin D47, Portland Cement Association, April 1961.

3. Di Stasio, J., and Van Buren, M. P., "Transfer of Bending Moment Between Flat Plate Floor and Column," ACI Journal, V. 57, No. 3, Sept. 1960, pp. 299-314.

4. *Commentary on Building Code Requirements for Reinforced Concrete* (ACI 318-63), American Concrete Institute.

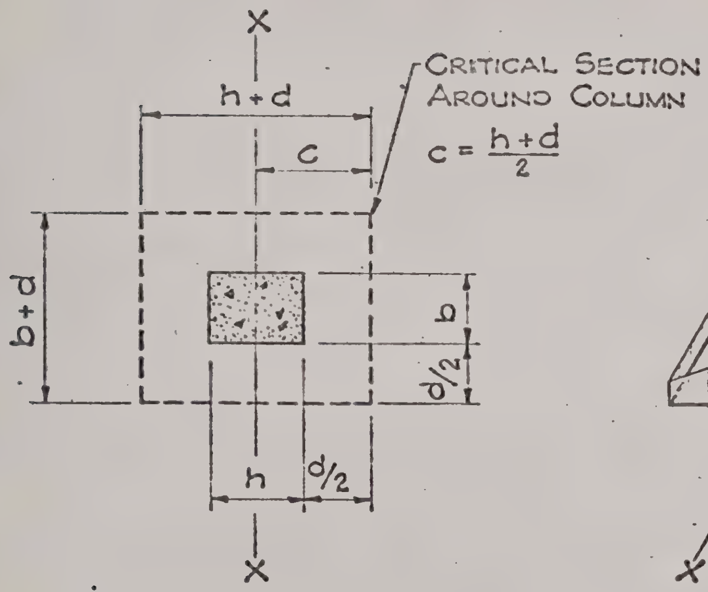
5. "Proposed Revision of ACI 318-63: Building Code Requirements for Reinforced Concrete," ACI Journal, V. 67, No. 2, February 1970.

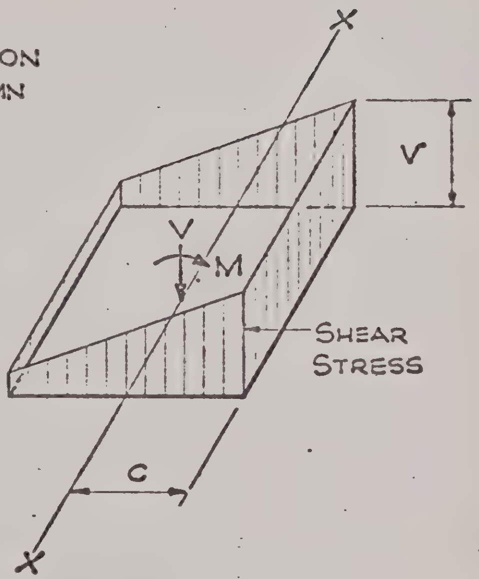
6. Mast, P.E., "Stresses in Flat Plates Near Columns," ACI Journal V. 67, No. 10, October 1970.

7. ACI-ASCE Committee 326, "Shear and Diagonal Tension," ACI Journal, V. 59, Nos. 1, 2, and 3, January, February, March 1962.

These references are representative of the published literature in this subject area. For the case where both vertical load and bending moment are transferred from a flat slab into a column, a simplified expression for determining the maximum slab shear stress at a cricial section around the column has been suggested:

$$v_{max.} = \frac{V}{A} + \frac{KMc}{J} \qquad (1)$$





Where V = Vertical Load Transmitted to Column

M = Bending Moment Transmitted to Column

b, h = Column Dimensions

d = Depth from Center of Gravity of Tensile Reinforcement in Slab to the Compression face.

K = Moment Reduction Factor, Expressed as a Fraction

J = Property of Slab Section Analagous to Moment of Inertia

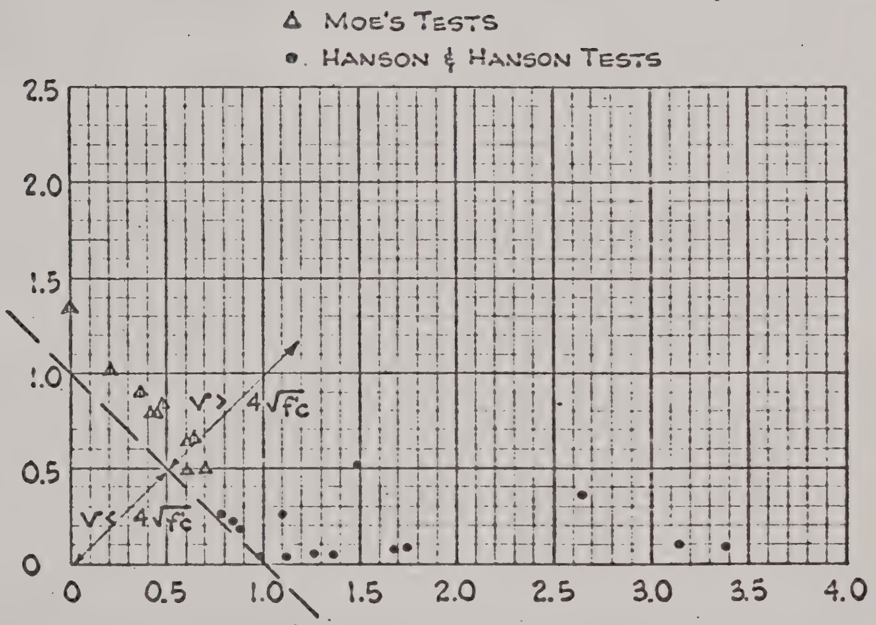
Several different values of K and different expressions for calculating J have been suggested. These differences have ocurred because different investigators have attempted to relate the same simplified formula to a phenomenon which is inherently more complex than the formula indicates. On the basis of correlating the series of ultimate load test results shown in the graph below with the simplified formula, the authors of reference 1 suggest that shear failures might be expected if:

$$v_{max.} > 4\sqrt{f'_c} \qquad (2)$$

WHEN $K = 0.4$

$$Jx = \frac{d(h+d)^3}{6} + \frac{(h+d)d^3}{6} + 2(b+c)dc^2$$

$$\frac{V_u}{v_u A} = \frac{V}{4\sqrt{f'_c}\,A}$$



$$\frac{\dfrac{M_u}{\dfrac{v_u J}{Kc}}}{} = \frac{M_u}{\dfrac{4\sqrt{f'_c}\,J}{0.4c}}$$

This conclusion with a speculative refinement for calculating the value of K

$$K_x = \left[1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b+d}{h+d}}} \right]$$

is included in the "ultimate load" method for the design of flat slabs as suggested in reference 5. The speculative refinement gives K = 0.4 only for square columns.

For the 12 x 24 inch column cross sections which are of concern in the 2000 Commonwealth Avenue structure, the refinement for K would give 0.34 or 0.46 depending upon the direction of bending. Such refinement does not seem justified in this case, especially since the effect would be more or less cancelled as a consequence of biaxial bending.

In this investigation the slab ultimate shear capacity around a given column is determined on the basis of Equations (1) and (2). If the compressive strength of the concrete is known, failure may be expected if the maximum shear stress calculated by Equation (1) exceeds the limit noted in Equation (2). Alternatively, the shear stress calculated according to Equation (1) may be inserted into Equation (2) to determine a minimum concrete compressive strength which would be needed to preclude failure. The following list notes required compressive strengths for a range of calculated maximum shear stresses.

Calculated v_{max} (psi)	Minimum Required Compressive Strength to Preclude Failure $f'_c = \frac{(v_{max})^2}{16}$ (psi)
155	1500
179	2000
200	2500
219	3000
237	3500
253	4000
268	4500

3. Results for East Side of Main Roof

Run 1

At Top of Column	V (kips)	M_z (kip ft.)	M_y (kip ft.)
3 (E2)	30.8	6.2	7.0
7 (E3)	34.9	4.6	-0.4
11 (E4)	37.8	10.8	-0.8
16 (E5)	72.9	4.7	22.2
15 (F5)	72.0	-6.5	21.3
21 East Wall Elevator Core	55.9	100.7	-90.5
22	16.5	-2.6	-70.6
23	16.2	3.0	-70.7
24	59.3	-111.9	-93.4

All perimeter joints deflect downward. These deflections range from 0.002 to 0.013 feet.

Runs 2, 2A, and 2B

At Top of Column	V (kips)	M_z (kip ft.)	M_y (kip ft.)
Run 2			
3 (E2)	17.6	2.9	28.2
7 (E3)	36.8	3.4	-2.9
11 (E4)	38.9	9.6	-0.4
16 (E5)	75.8	8.2	22.0
15 (F5)	74.9	-10.0	21.2
21 East Wall Elevator Core	57.1	104.3	-92.2
22	16.4	2.8	-71.4
23	16.1	3.1	-71.5
24	60.5	-115.5	-95.2
Run 2A			
3 (E2)	17.6	2.9	28.3
7 (E3)	37.1	3.6	-2.3
11 (E4)	40.7	11.4	-0.2
16 (E5)	77.5	10.0	22.3
15 (F5)	76.6	-11.8	21.5

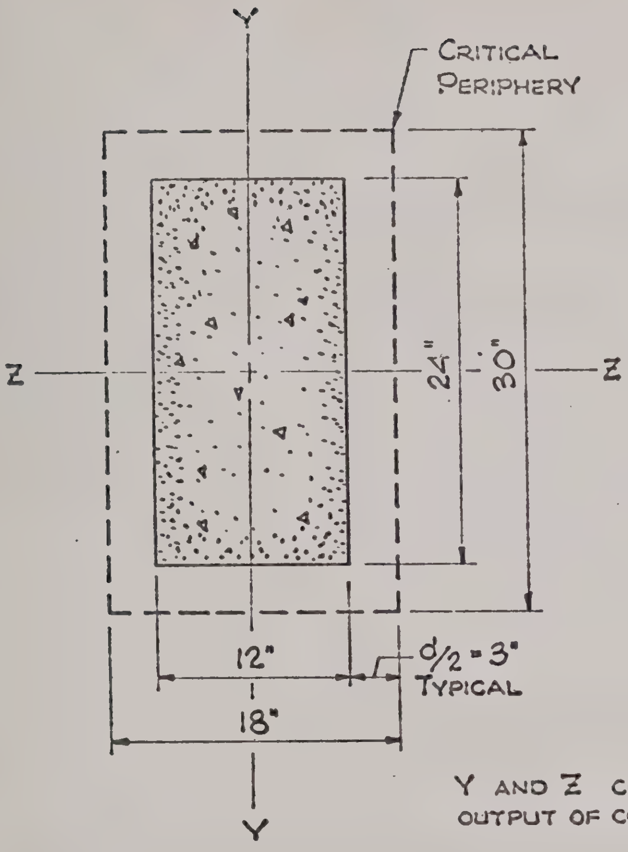
At Top of Column	V (kips)	M_z (kip ft.)	M_y (kip ft.)
Run 2B			
1 (E2)	17.2	4.6	29.8
3 (E3)	37.2	7.1	-2.5
5 (E4)	39.9	14.6	-0.9
7 (E5)	77.7	16.1	23.7

In Runs 2, 2A, and 2B all perimeter joints deflect upward. These deflections range from 0.001 to 0.014 feet. This fact shows that perimeter reshoring would not be effective for helping to carry superimposed loads on the interior portions of the main roof.

Also it should be noted that the axial loads and bending moments induced at the tops of the interior columns are not significantly influenced by loads on or shores under the overhanging slabs.

Comparison of results from Run 2A (coarse grid) and Run 2B (fine grid) indicates that the coarse grid gives reasonably accurate results. The most significant difference in the result is for the M_z bending moments induced in the E line columns. The Run 2B results are felt to be more accurate than those of Run 2A, principally because the fine grid does not contain very rectangular elements in critical areas and also because it has smaller elements.

For the vertical shear and moments calculated in Run 2B, the maximum slab shear stress on a peripheral section out 3 inches from column E5 is computed on the following page.



SLAB THICKNESS = $7\frac{1}{2}$"

d = 6"

$$A = 2(6)(18+30) = 576 \text{ IN}^2$$

$$J_Y = \frac{6(18)^3}{6} + \frac{18(6)^3}{6} + 2(30)(6)(9)^2$$

$$= 5832 + 648 + 29160$$

$$= 35640 \text{ IN}^4$$

$$J_Z = \frac{6(30)^3}{6} + \frac{30(6)^3}{6} + 2(18)(6)(15)^2$$

$$= 27000 + 1080 + 48600$$

$$= 76680 \text{ IN}^4$$

Y AND Z COORDINATE AXES ARE CONSISTENT WITH OUTPUT OF COMPUTER RESULTS FOR EAST SIDE ANALYSES

$$v_{max} = \frac{V}{576} + \frac{0.4\,M_Y(9)}{35640} + \frac{0.4\,M_Z(15)}{76680}$$

$$= \frac{77.7}{576} + \frac{0.4(23.7\times12)(9)}{35640} + \frac{0.4(16.1\times12)(15)}{76680}$$

$$= 0.135 + 0.029 + 0.015$$

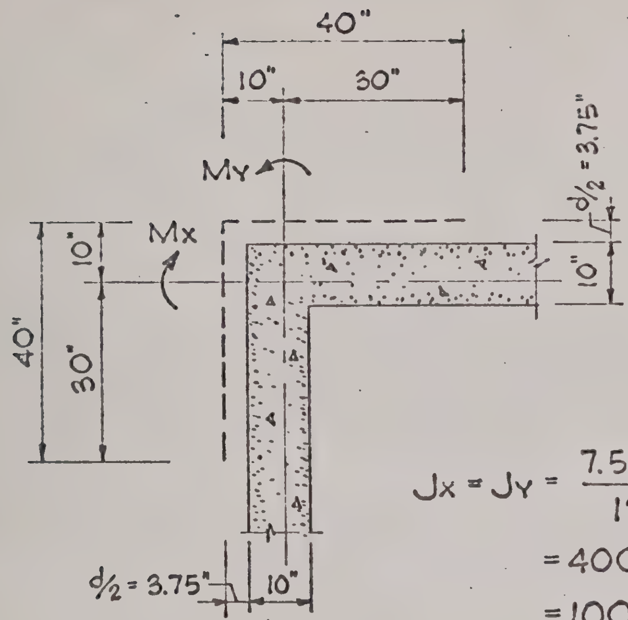
$$= 0.179 \text{ KSI}$$

Figures 1.3a through 1.3g contain plots of slab bending moments obtained from Run 2B. Figure 1.2 keys these several moment diagrams.

Also shown on Figures 1.3a through 1.3g by horizontal lines are ultimate flexural capacities of the designed slab. Comparison of these capacities with the calculated moments indicates that yielding at the east face of the elevator core must have occurred.

Not only does the analysis of Run 2B reveal a flexural overload along the east face of the elevator core wall, but also the shear condition in the roof slab at the corners of the core is severe. This condition arises because of the transfer of vertical load at the corners in addition to the transfer of slab bending moments into the core walls. In Run 2B the largest slab bending moments in the entire roof slab occur at the southeast corner of the core. Bending moments in this region are transferred into the core walls by shearing stresses in the same way as bending moments are transferred into column E5. The magnitude of the vertical load transferred at the southeast corner is indicated by the axial load (60.5 kips) calculated in Run 2 for the replacement column at the southeast corner.

Assume at the southeast corner of the elevator core that the following section could transmit vertical shear and bending moment:



SLAB THICKNESS = 9"

$d = 7\frac{1}{2}"$

MX AND MY ARE SLAB BENDING MOMENTS

$$J_X = J_Y = \frac{7.5(40)^3}{12} + 7.5(40)(10)^2 + 7.5(40)(10)^2$$

$$= 40000 + 30000 + 30000$$

$$= 100000 \text{ IN}^4$$

$$A = 80(7.5) = 600 \text{ IN}^2$$

$$v = \frac{V}{A} + \frac{M_X Y}{J_X} + \frac{M_Y X}{J_Y}$$

$$v_{max} = \frac{60.5}{600} + \frac{M_X(10)}{100000} + \frac{M_Y(10)}{100000}$$

Assuming a maximum available ultimate shear stress of 0.2 ksi at the corner of the critical sections, failure would be expected if

$$M_X + M_Y > \frac{0.200 - 0.101}{0.0001} = 990 \text{ KIP INCHES}$$

$$\doteq 82.5 \text{ KIP FEET}$$

A comparison of this calculation with the bending moments M_x and M_y in element 150 of Figures 1.3f and 1.3b, respectively, (i.e., Run 2B using the grid in Figure 1.2) indicates that slab shear stresses at the corner of the elevator core were nearly as severe as those at column E5.

The foregoing analyses of Runs 1, 2, 2A, and 2B led to the analysis of Run 3. With the limiting assumption that the roof slab is free to rotate (hinged) at the face of the elevator core wall, the resulting forces induced into the top of column E5 are, predictably, increased.

Run 3

At Top of Column	V (kips)	M_z (kip ft.)	M_y (kip ft.)
3 (E2)	30.7	6.2	6.7
7 (E3)	35.3	4.8	-0.02
11 (E4)	38.6	12.9	-1.9
16 (E5)	84.7	15.2	34.1
15 (F5)	83.3	-17.8	32.6
21	69.4	0	0
22 East Wall	3.7	-13.0	0
23 Elevator	2.3	14.5	0
24 Core	74.6	0	0

In accord with the previous discussion (under Runs 2, 2A, and 2B) regarding accuracy of the results from a coarse grid analysis, it is felt that the M_z bending moment results for Run 3 are too small. Instead of 15.2 kip feet for column E5, for example, a moment of 20 kip feet seems more reasonable.

With these conditions, the maximum shear stress along the critical section around column E5 would be

$$V_{max.} = \frac{64.7}{576} + \frac{0.4(34.1 \times 12)(9)}{35640} + \frac{0.4(20 \times 12)15}{76680}$$

$$= 0.147 + 0.041 + 0.019$$

$$= 0.207 \text{ KSI}$$

With a maximum shear stress of about 179 psi (Run 2B) or of about 207 psi (Run 3 above) failure might be expected if the compressive strength were below 2000 or 2600 psi, respectively.

Run 4 (Remove column E5 from Run 3)

At Top of Column	V (kips)	M_z (kip ft.)	M_y (kip ft.)
3 (E2)	31.2	5.9	8.2
7 (E3)	29.9	4.0	-10.1
10 (F4)	39.7	-26.9	3.4
11 (E4)	67.7	17.7	50.3
15 (F5)	106.9	-96.3	36.6
21 East Wall Elevator Core	70.2	0	0
22	4.8	-15.4	0
23	1.5	17.2	0
24	105.5	0	0

At the critical section around column 15 (column F5):

$$V_{max.} = \frac{106.9}{576} + \frac{0.4(36.6 \times 12)(9)}{35640} + \frac{0.4(96.3 \times 12)(15)}{76680}$$

$$= 0.185 + 0.044 + 0.090$$

$$= 0.319 \text{ KSI}$$

In addition to throwing off load to columns 24, 15, etc., loss of column 16 (E5) caused the slab bending moments about the E and 5 lines to change dramatically. Whereas in Run 2B the slab moments about line E between column E5 and the elevator core all indicate tension in the top surface of the slab (see Figures 1.3d, 1.3e, and 1.3f), the results of Run 4 indicate that these moments change sign so that large tension stresses would be induced in the bottom surface. As designed, the north-south bottom steel was interrupted at line E. These facts explain the crack referred to by Mr. Fantasia (which he said ran from column E5 toward column D8) or the belly referred to by Mr. Feudo (which he said ran from the southeast corner of the elevator core toward column E5).

Run 5

At Top of Column	V (kips)	M_z (kip ft.)	M_y (kip ft.)
3 (E2)	31.6	5.9	9.5
7 (E3)	24.9	1.3	-18.9
10 (F4)	83.0	-46.5	81.7
11 (E4)	83.5	45.7	82.1
24 Southeast Corner Elevator Core	120.0	0	0

The only purpose in making the Run 5 analysis was to give indication of the spread of load to columns E4 and F4 which might be anticipated. The forces induced at columns E4 and F4 certainly are large enough to indicate the possibility of shear failure at these locations, but also the moments induced at several places (see computer output) are much larger than the flexural capacity of the slab.

The state of the art does not contain any procedure for analytically determining the extent of lateral failure propogation in a flat slab structure in the event of shear failure at one column.

B. EAST SIDE OF SIXTEENTH FLOOR

The distribution of load imposed on the 16th floor after collapse of the roof is not known exactly. Assuming that the entire roof slab came to rest statically from line 2 to line 6, the previously computed forces and stresses would be superimposed on the self-weight forces and stresses in the 16th floor slab.

A finite element analysis was not made for the self-weight condition. The self-weight induced forces at column E5 were approximated by making rigid frame analyses along lines 5 and E. The calculated moments were then reduced to account for the fact that a rigid frame analysis will give higher column moments than a finite element analysis. These calculations indicated for the top of the 15th story column E5:

$$V_{\text{self-weight}} = 37 \text{ kips}$$

$$M_{z_{\text{self-weight}}} = 10 \text{ kip ft.}$$

$$M_{y_{\text{self-weight}}} = 4 \text{ kip ft.}$$

$$v_{\text{max.}*} = \frac{37}{576} + \frac{0.4(4\times 12)(9)}{35640} + \frac{0.4(10\times 12)(15)}{76680}$$

$$= 0.064 + 0.005 + 0.009$$

$$= 0.078 \text{ KSI}$$

* SELF-WEIGHT AT COLUMN E5

This self-weight slab shear stress would be added to the value of 0.179 ksi (Run 2B) or 0.207 ksi (Run 3) calculated for the roof slab, giving

$$0.257 \text{ KSI} < v_{max.}, \text{ COLUMN E5, } 16^{TH} \text{ FLOOR} < 0.285 \text{ KSI}$$

Even with $v_{max} = 257$ psi failure might be expected if the concrete compressive strength was below 4200 psi.

It must be understood that the lateral propagation of collapse at the 16th floor level occurred under quite different conditions than had been present in the main roof. The loading, the f'_c, the quantity and location of reinforcement, the continuity of the columns, etc. were all different. Analyses similar to those made for the roof slab would show, successively, the development of large forces near columns adjacent to a failed area.

C. WEST WIDE OF MAIN ROOF

Figures 1.4 and 1.5 show the finite element grids which were used for the analysis of the west half of the main roof. The grid in Figure 1.4 incorporates the same number of elements and columns as did the east side grid of Figure 1.1. The grid in Figure 1.5 was used to determine whether a change in grid size would significantly influence the results obtained for column forces. Since the loading assumed for the west side was symmetrical, the finer grid of Figure 1.5 (with appropriate symmetry boundary conditions) and the relatively coarse grid in Figure 1.4 represent an identical problem. The finer grid would give more accurate results.

Several grid lines in Figure 1.4 are changed from their east side positions. In this case columns 5, 8, 10 (E9), 11 (F9), 15(E8), 16(F8), and 19 through 32 are correctly located, as are the mechanical room floor slab, wall beams, walls, and brackets. As with the east side grid, however, there are some minor

discrepancies between the geometry of the grid and the actual slab.

Under the west half of the roof there was a line of shoring between lines 7 and 8 (see Part II, Section 7). These shores have been accounted for in the analysis by placing columns under joints 104, 106, 107, 108, 109, and 111 in the coarse grid and the equivalent joints in the fine grid. The modulus of elasticity of the 4 inch x 4 inch wood shores is about 1500 ksi, or 75% of that assumed for the concrete columns. In fact, however, the effective stiffness of the shores is influenced by the flexibility of the 16th floor slab, which in turn was reshored to the 15th floor in this general area. Since the stiffnesses of the shores are mutually dependent on one another, a precise evaluation of each effective stiffness would be very difficult.

On the basis of a simplified calculation the effective modulus was estimated to be about 550 ksi, or 28% of that assigned to the concrete columns. To assess the sensitivity of the results to differing effective stiffnesses, analyses were made incorporating effective moduli of 370 ksi and 740 ksi. Both the coarse and fine grids were analyzed using 370 ksi. These are identified as Runs 6 and 6A, respectively. Only the coarse grid was analyzed using 740 ksi and it is identified as Run 7.

In the analyses it was assumed that the mechanical room slab, walls, wall beams, and brackets had been poured up to the east wall of the elevator core. The two boilers were not included in the loading as their presence would have only a tiny influence on the forces around columns [illegible] 9. Although it is known that a few kips of miscellaneous construction materials were lying on the south side of the roof between line 7 and 11, they were not included in the analyses.

Results Run 6

Coarse grid Figure 1.4

$E_{\text{effective, wood shores}} = 370$ ksi.

At Top of Column		V (kips)	M_z (kip ft.)	M_y (kip ft.)
10 (E9)		85.4	-28.1	16.9
15 (E8)		90.9	-18.7	-44.1
21	West Wall Elevator Core	43.3	79.9	-59.1
22		11.9	-1.5	-46.2
23		11.9	1.5	-46.2
24		43.3	-79.9	-59.1
35	Shores	8.7	0	0
36		7.9	0	0
37		7.6	0	0

Results Run 6A

Fine grid Figure 1.5

$E_{\text{effective, wood shores}} = 370$ ksi

At Top of Column		V (kips)	M_z (kip ft.)	M_y (kip ft.)
6 (E9)		83.6	-32.1	29.3
9 (E8)		92.2	-22.9	-20.3
11	West Wall Elevator Core	38.0	62.0	-42.5
12		9.3	-1.5	-27.7
18	Shores	10.1	0	0
19		9.0	0	0
20		10.6	0	0

Results Run 7

Coarse grid Figure 1.4

$E_{\text{effective, wood shores}} = 740$ ksi

At Top of Column		V (kips)	M_z (kip ft.)	M_y (kip ft.)
10 (E9)		85.7	-28.2	17.4
15 (E8)		88.4	-18.4	-49.6
21	West Wall Elevator Core	38.2	73.3	-46.5
22		9.5	-1.3	-35.4
23		9.5	1.3	-35.4
24		38.2	-73.3	-46.5
35	Shores	12.6	0	0
36		11.7	0	0
37		10.9	0	0

All Results Symmetrical

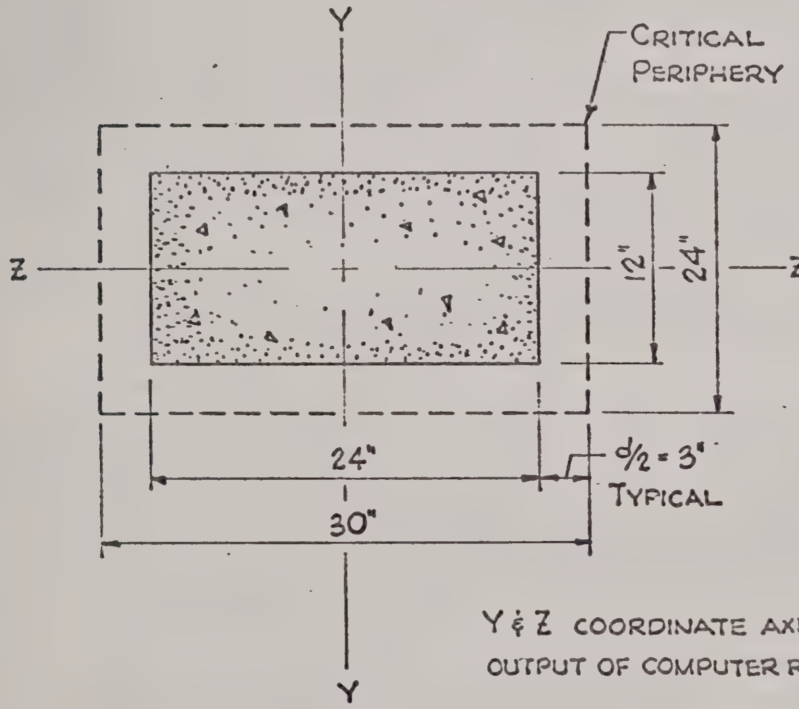
Several observations regarding the results deserve comment. First, the discrepancies between the coarse and fine grid analyses are greater for the west side analyses than was the case for the east side analyses. The shifting of grid lines for the west side coarse grid resulted in an additional loss of "squareness" in the elements. Accordingly, the accuracy of the coarse grid west side runs is not nearly as good as that of the fine grid west side analysis.

Independent of the above consideration, the analyses show that forces induced in the slab and columns are not drastically influenced by a wide difference in the effective stiffness of the wood shores.

As a consequence of these shores, the M_y slab bending moments adjacent to the west wall of the elevator core are only about one-half of the magnitude of the

slab moments adjacent to the east wall determined in the east side analyses. Even though rotational yielding in the slab would not be expected (inspection verified that it did not occur), the slab conditions around columns E8, F8, E9, and F9 are still severe.

Columns E8, F8, E9 and F9



SLAB THICKNESS = $7\frac{1}{2}$"
d = 6"

$A = 576 \text{ IN}^2$
$J_Y = 76680 \text{ IN}^4$
$J_Z = 35640 \text{ IN}^4$

Y & Z COORDINATE AXES ARE CONSISTENT WITH OUTPUT OF COMPUTER RESULTS FOR WEST SIDE ANALYSES

At E8 and F8 **Run 6A**

$$V_{max} = \frac{92.2}{576} + \frac{0.4(22.9\times12)(9)}{35640} + \frac{0.4(20.3\times12)(15)}{76680}$$

$$= 0.160 + 0.028 + 0.019$$

$$= 0.207 \text{ KSI}$$

At E9 and F9 Run 6A

$$
\begin{aligned}
V_{max} &= \frac{83.6}{576} + \frac{0.4(32.1 \times 12)(9)}{35640} + \frac{0.4(29.3 \times 12)(15)}{76680} \\
&= 0.145 + 0.039 + 0.027 \\
&= 0.211 \text{ ksi}
\end{aligned}
$$

These stresses are slightly higher than those calculated for a section around column E5 on the east side. Figure 1.6 shows photographs of the load induced cracks on the top surface of the roof slab above columns E8 and F8. Hairline crack patterns appear over columns E9 and F9 but they do not appear to be serious.

D. COMMENTS REGARDING COMPUTER PROGRAM

A finite element solution of a continuous equilibrium problem consists of the following basic steps:

1. Derivation for each element of the stiffness matrix relating nodal forces and nodal displacements (similar to the stiffness matrix of a linear member).

2. Derivation for each element of equivalent nodal forces corresponding to distributed loads over the element (or part of it). This is equivalent to the determination of fixed end forces at the ends of a beam with a distributed load.

3. Assembly of the total stiffness matrix for the selected mesh of finite elements, equivalent to the assembly of the matrices of individual beam and columns to obtain the total stiffness matrix of a frame.

4. Assembly of the total vector of nodal forces including concentrated loads directly applied at the nodes and the equivalent nodal forces resulting from distributed loads over the elements.

5. Modification of the total stiffness matrix and the load vector to account for support conditions (displacements prevented at some of the nodes in specified directions).

6. Solution of a system of linear equations where the matrix of coefficients is represented by the modified total stiffness matrix. The right hand side of the equations is the modified vector of nodal forces, and the unknowns are the nodal displacements. In general this is a very large system of equations requiring the use of a digital computer for its solution. (In Run 1, for example, we had 210 nodes with 6 unknown displacements per node or a system of 1260 equations with 1260 unknowns.)

7. Once the nodal displacements are known each element is again considered separately and from the assumed displacement expansion through its domain, internal forces can be evaluated at any point. It is important to notice that, when displacements are the primary variables, moments and shears are obtained by computing derivatives of the displacement functions and their accuracy is smaller. While internal forces can be obtained at any point of the element, if they are determined at the nodes the values resulting from different elements will not be the same. It is possible to average these different values, but in general it is preferred to determine the internal forces at the centroid of each element. This was the approach followed in the present case.

The formulation of a plate problem, according to the Theory of Elasticity, includes two phases, which, for small deflections, can be considered uncoupled:

1. Determination of the in plane displacements and forces (N_x, N_y, $N_{xy} = N_{yx}$) at the middle surface of the plate. Mathematically this problem is a plane stress problem and is normally referred to as the plate stretching problem. The solution can be obtained by using plane stress rectangular elements. The simplest one of these elements uses a quadratic expansion for the displacements (u in the x direction, v in the y direction) and has two degrees of freedom, u and v, at each node. The PSR element of the STRUDL program, for example, is such an element.

   When the plate is connected to linear members (beams or columns) it is preferable to have a cubic expansion and a third degree of freedom at each node representing the rotation around an axis perpendicular to the plane of the plate (ϕ_z). A finite element with these characteristics was developed at M.I.T. and implemented in the reinforced concrete phase of STRUDL (it is not, however, available in the finite element capabilities of STRUDL). This type of finite element was used in the present analyses for the in-plane displacements.

2. Determination of the transverse displacements w (perpendicular to the plane of the slab) and the moments M_x, M_y, and M_{xy}. This is normally referred to as the plate bending problem. A variety of rectangular elements have been developed for the solution of this problem. The simplest one (first order rectangle) assumes a cubic expansion for the transverse displacement w and has three degrees of freedom per node (the displacement w and the rotations around the x and y axis, ϕ_x and ϕ_y). It satisfies inter-element compatibility with respect to displacements but not with respect to the normal slope. It has been shown, however, to converge to the exact solution and to behave satisfactorily for practical purposes. Another element, which is fully compatible with respect to both displacements and rotations, has been suggested, but it does not include the constant

twist terms and does not converge to the exact solution. For the purposes of these analyses the first element was selected. It is the same element implemented in the STRUDL program (BPR element).

By combining the plane stress and plate bending elements an element is obtained with 6 degrees of freedom per node, three displacements (u, v, w) and three rotations (ϕ_x, ϕ_y, ϕ_z). The displacement expansions in the three displacements u, v, w are cubic (same as for linear members) and the element is therefore appropriate for the study of combinations of plates with beams and columns.

The computer program used for the analyses can consider any combination of the above mentioned elements and linear members in space. Input to the program are:

The number of nodes and number of elements.

A set of values for element thickness, modulus of elasticity, and Poisson's ratio which may apply to the majority of the elements.

The coordinates of the nodes in a global system of coordinate axes and for each support a code indicating prevented displacements.

The connectivity of each element (numbers of its nodes) and its thickness, modulus of elasticity and Poisson's ratio if different from the general ones specified at the beginning. If any of these values is specified as zero the general values are attributed to the element.

The number of linear members and number of additional joints if any.

The coordinates of the additional joints and a support code if appropriate.

A set of general values for modulus of elasticity and shear modulus of the linear members if common to several of them.

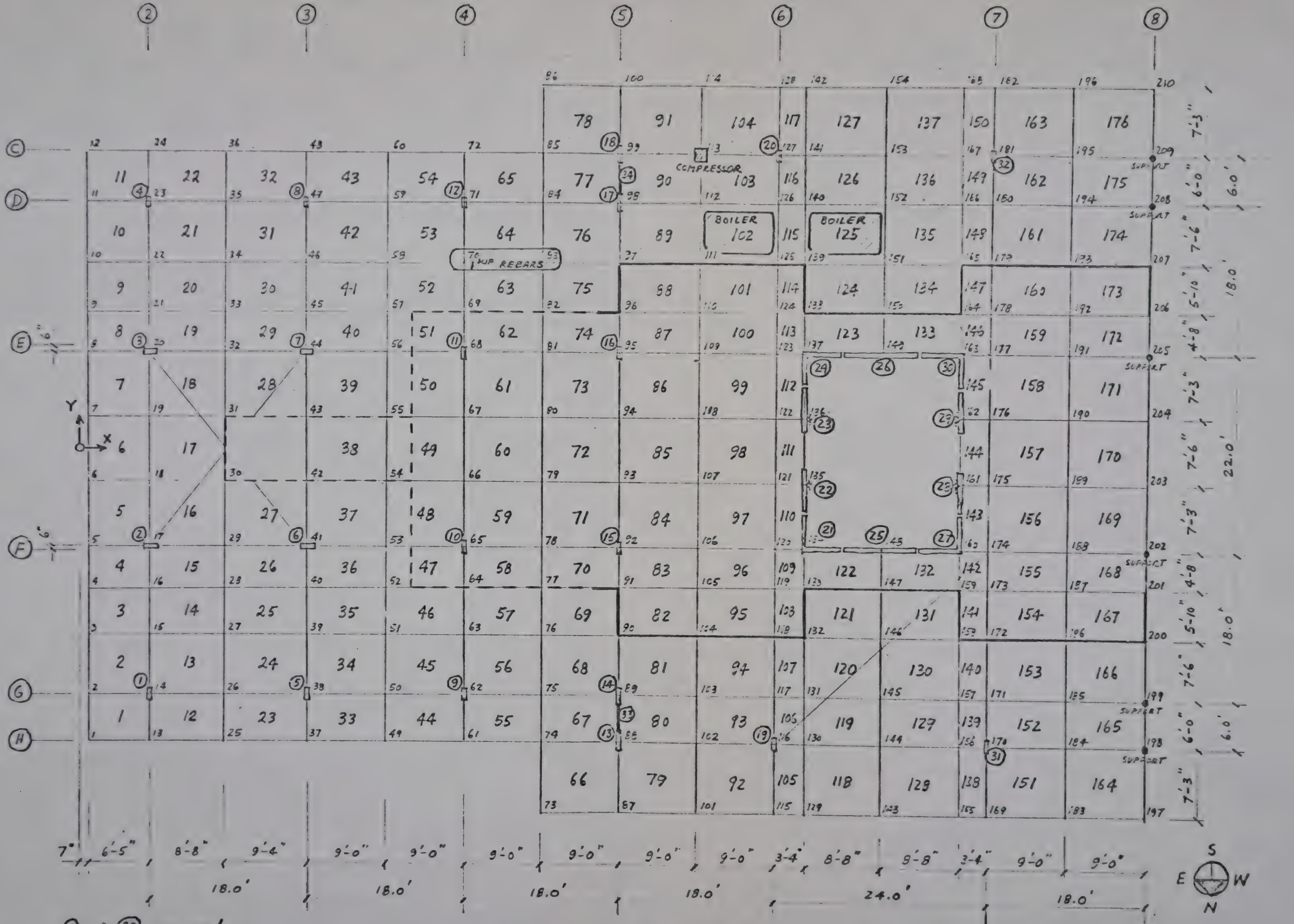
For each linear member its connectivity (numbers of the joints at start and end of member), its cross sectional area and moments of inertia, and the modulus of elasticity and shear modulus if different from the general values.

The number of concentrated loads applied at the nodes or joints.

For each concentrated load the number of the joint where it is applied and its components (FX, FY, MZ, FZ, MX, MY).

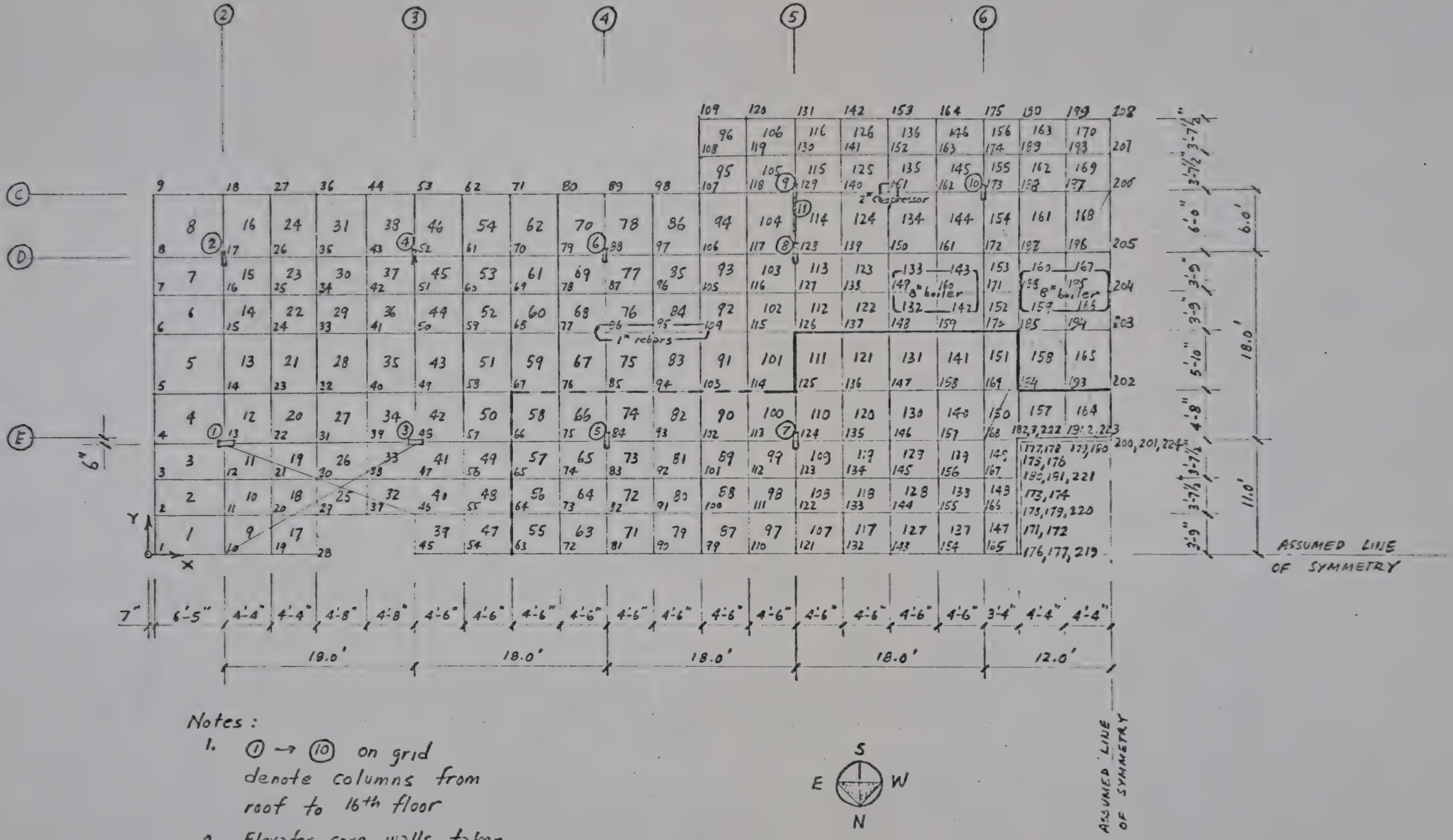
The number of distributed element loads and for each one of them the element over which it extends and its intensity.

The computer outputs contained in Appendix II.3 include the program input followed by some information on the way it is internally stored (for each element the connectivity, thickness, modulus of elasticity, Poisson's ratio, the values assigned internally to these variables, and the computed dimensions of the element are printed). This data is then followed by some additional information on the internal storage of the stiffness matrices and the use of disk for the benefit of the programmer. Finally, the results are printed consisting of joint displacements, stress resultants (NX, NY, NXY, MX, MY, MXY) at the centroid of each element, and forces at the ends of each linear member. Moments in the elements are positive when creating tensile stresses in the top face of the slab. The forces in the members are: FX axial force, FY, FZ shears, MX torsional moment, MY, MZ bending moments. They refer to local member axes: the local X axis being the longitudinal axis of the member, the local Y axis for the columns being parallel to the global Y axis, and the local Z axis being parallel to the global X axis.



① → ㉜ on grid denote columns from roof to 16th floor

Figure 1.1 FINITE ELEMENT "COARSE" GRID FOR EAST SIDE ROOF ANALYSES



Notes:

1. ① → ⑩ on grid denote columns from roof to 16th floor
2. Elevator core walls taken as elements 171 → 188

Figure 1.2 FINITE ELEMENT "FINE" GRID FOR EAST SIDE ROOF ANALYSIS

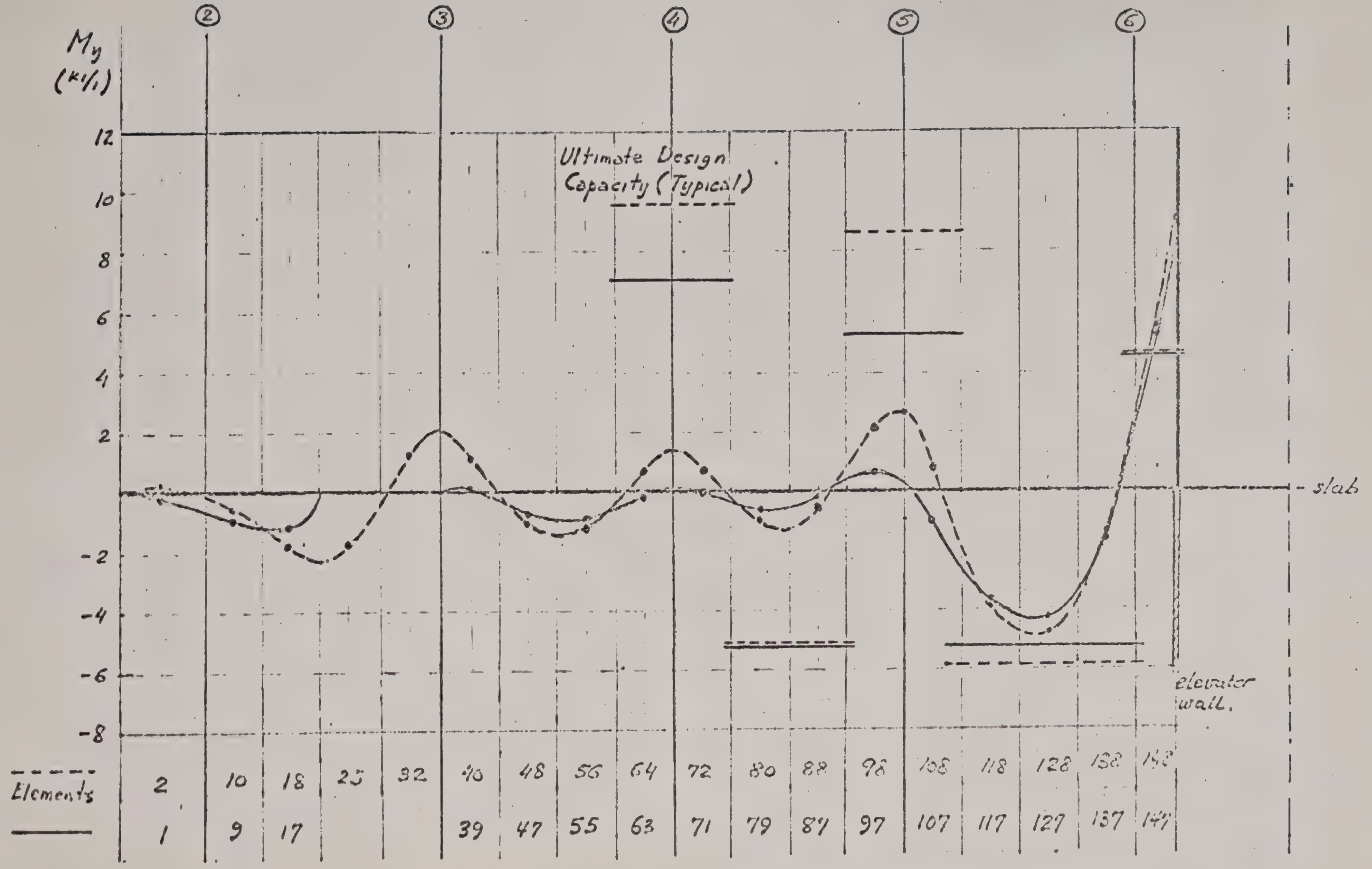


FIGURE 1.3a SLAB BENDING MOMENT DIAGRAMS FROM RUN 2B, FIGURE 1.2 GRID

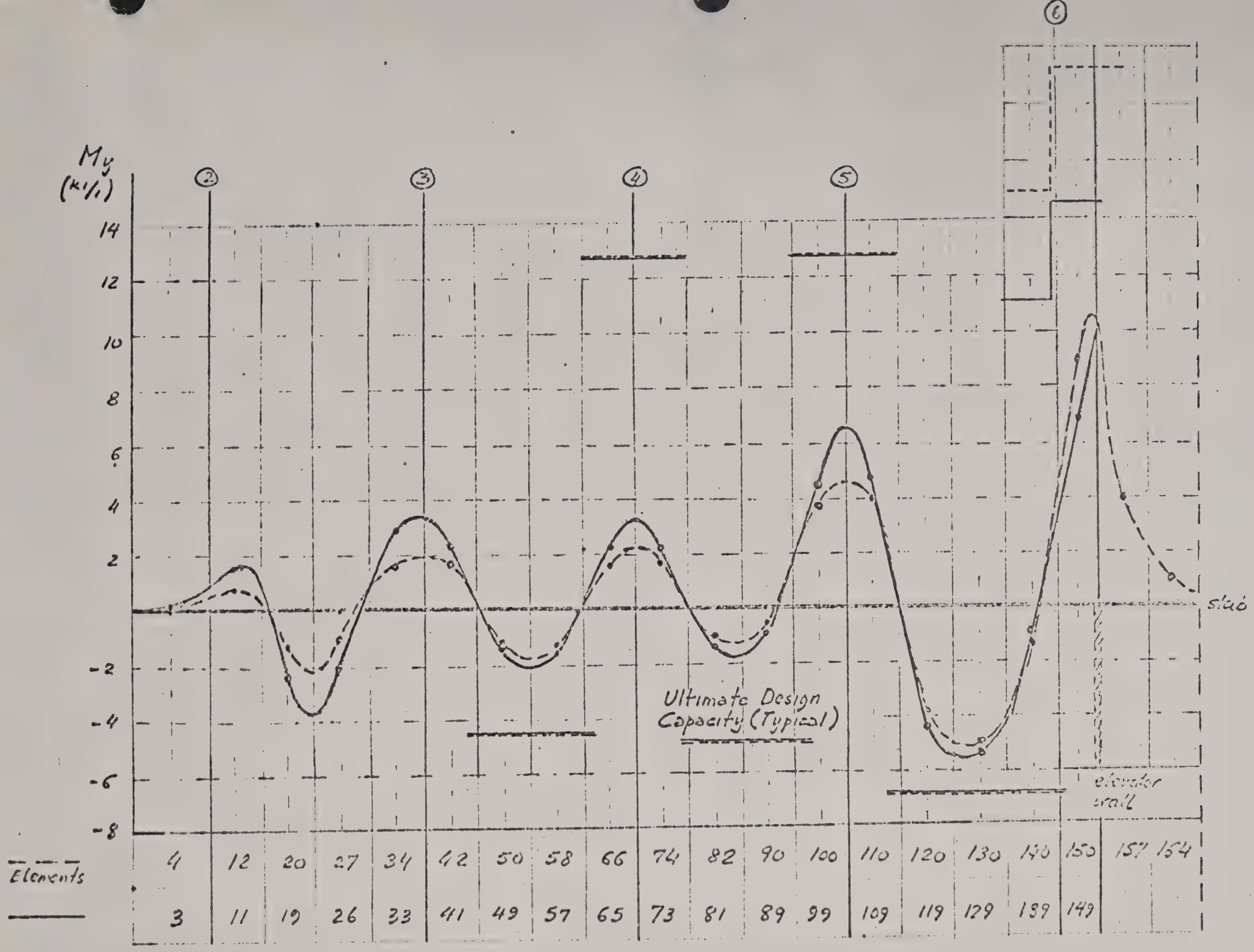


FIGURE 1.3b SLAB BENDING MOMENT DIAGRAMS FROM RUN 2B, FIGURE 1.2 GRID

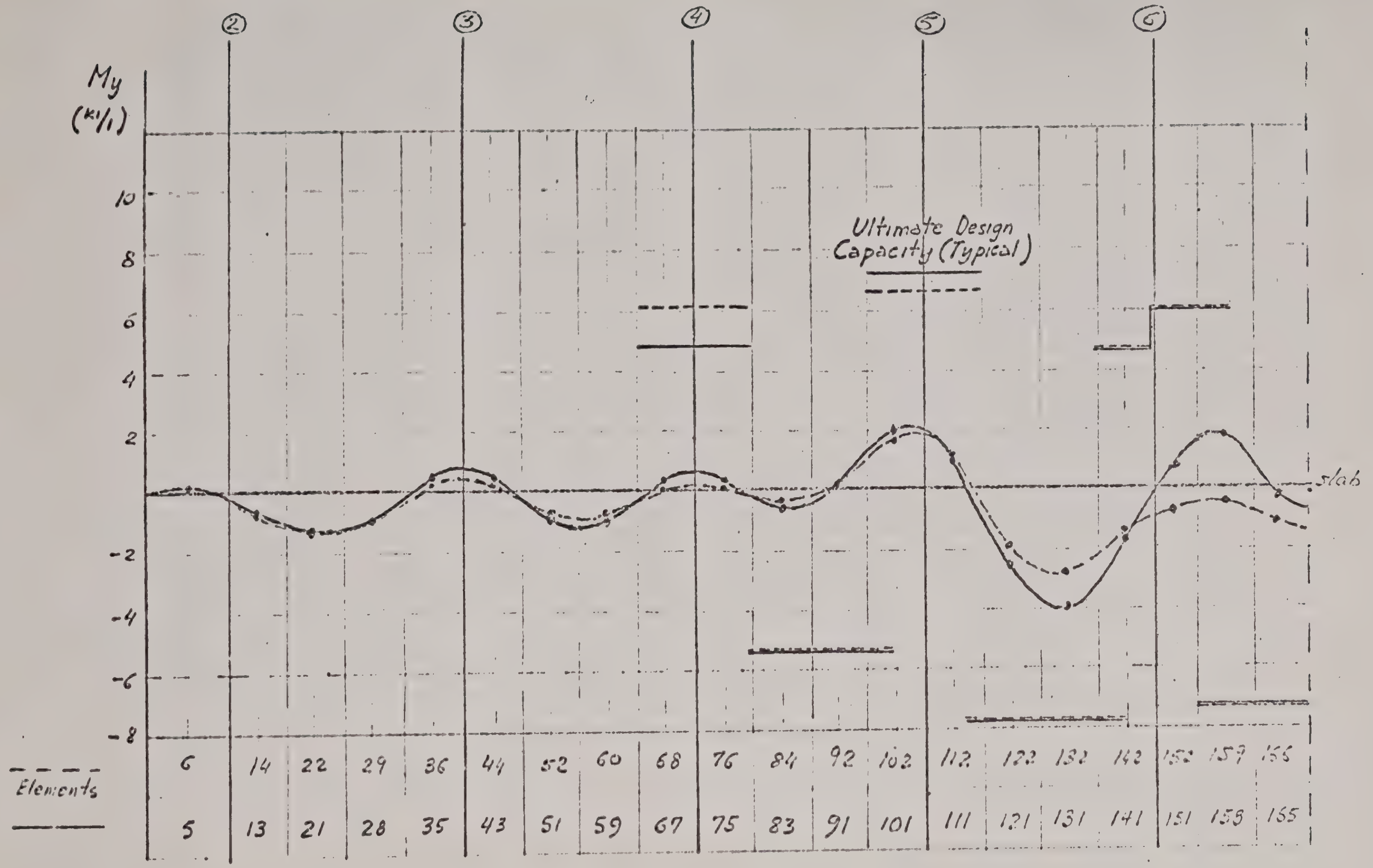


FIGURE 1.3c SLAB BENDING MOMENT DIAGRAMS FROM RUN 2B, FIGURE 1.2 GRID

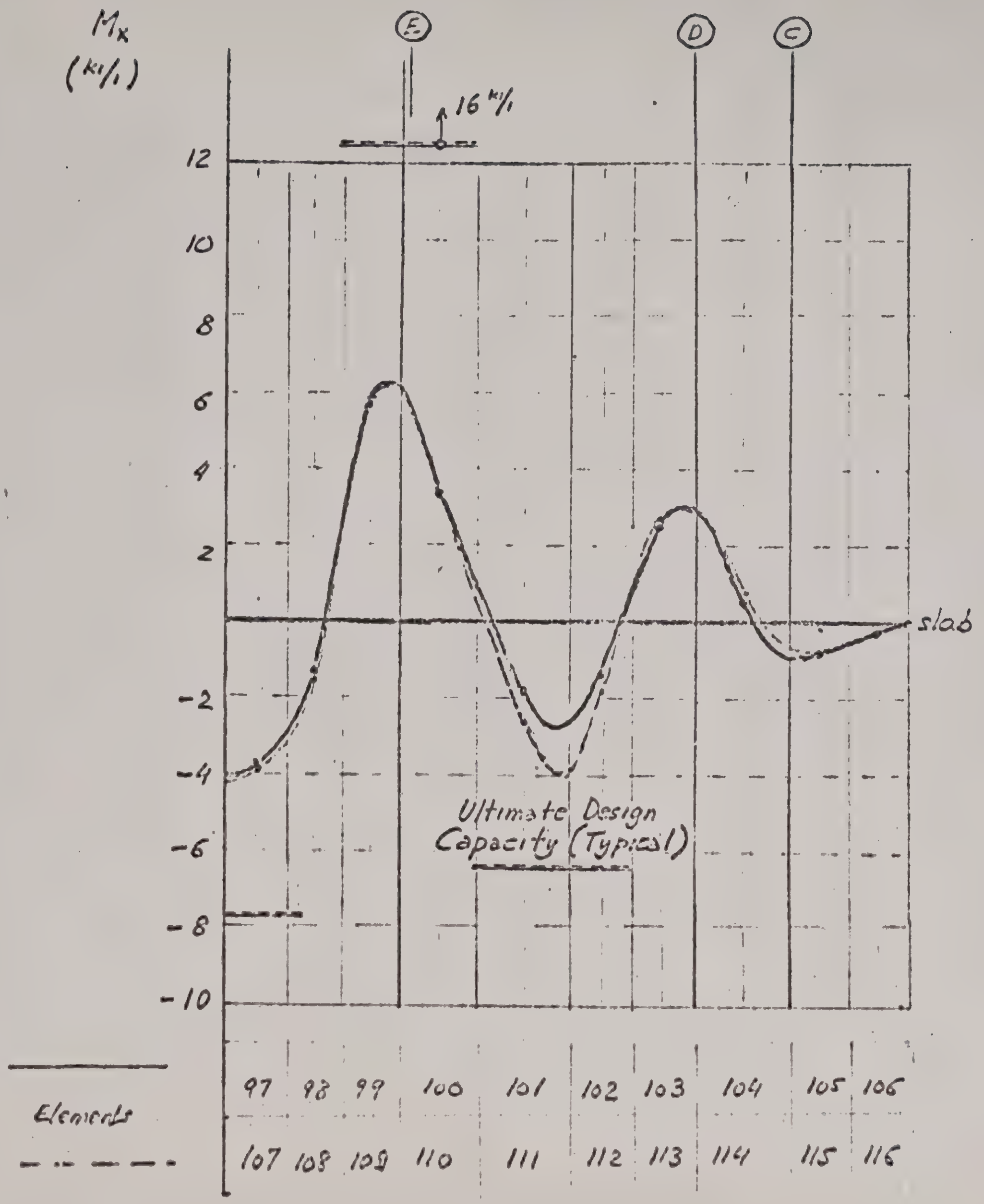


FIGURE 1.3d SLAB BENDING MOMENT DIAGRAMS FROM RUN 2B, FIGURE 1.2 GRID

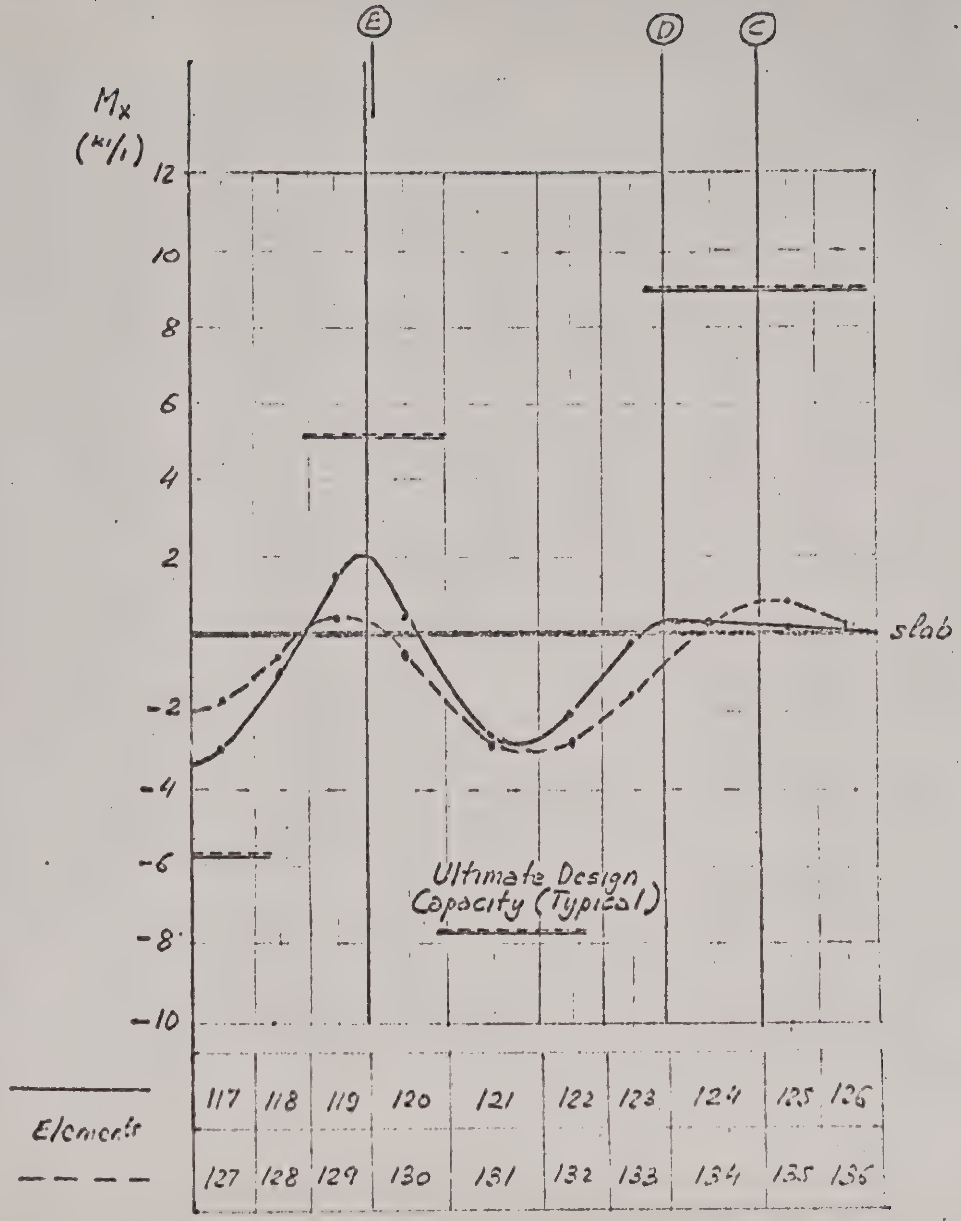


FIGURE 1.3c SLAB BENDING MOMENT DIAGRAMS FROM RUN 2B, FIGURE 1.2 GRID

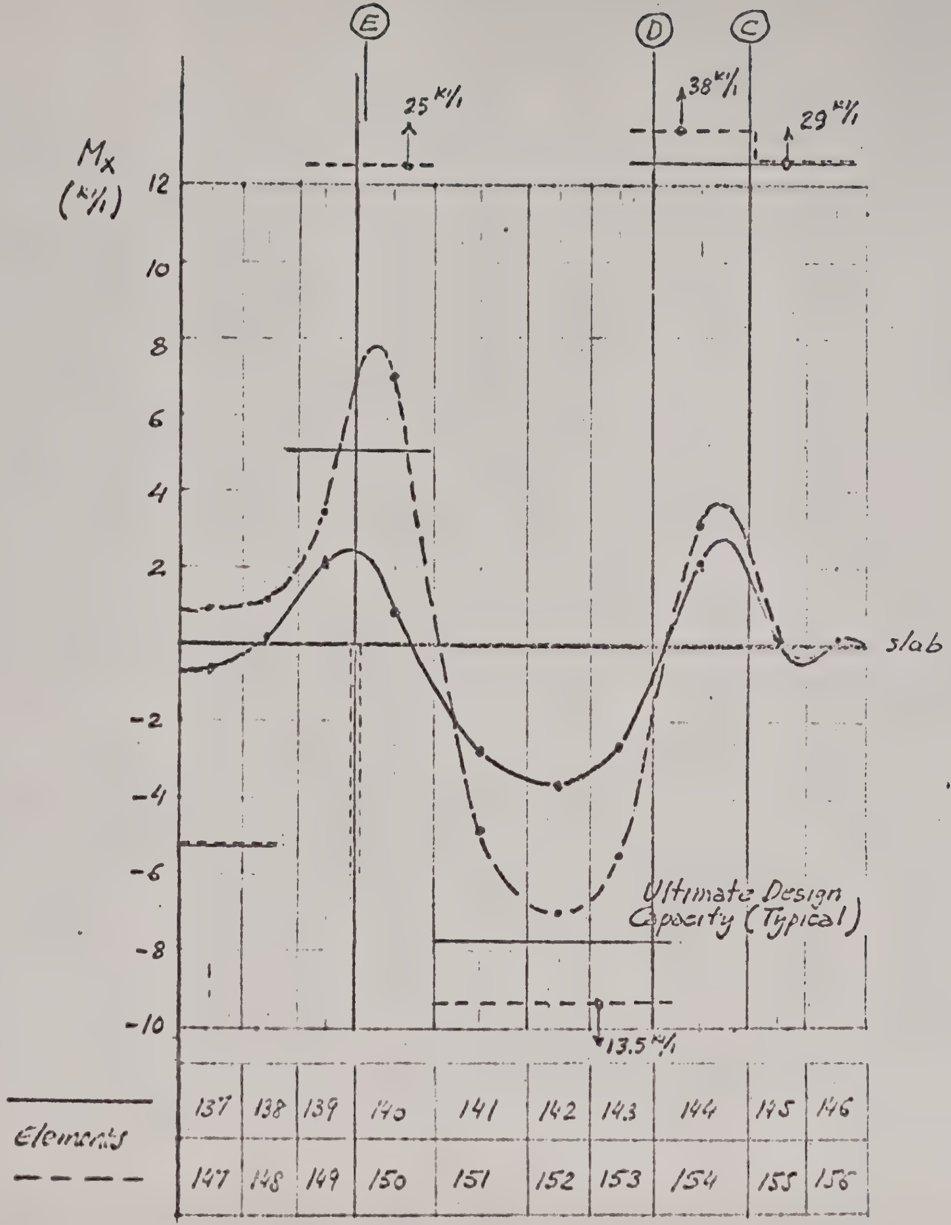


FIGURE 1.3f SLAB BENDING MOMENT DIAGRAMS FROM RUN 2B, FIGURE 1.2 GRID

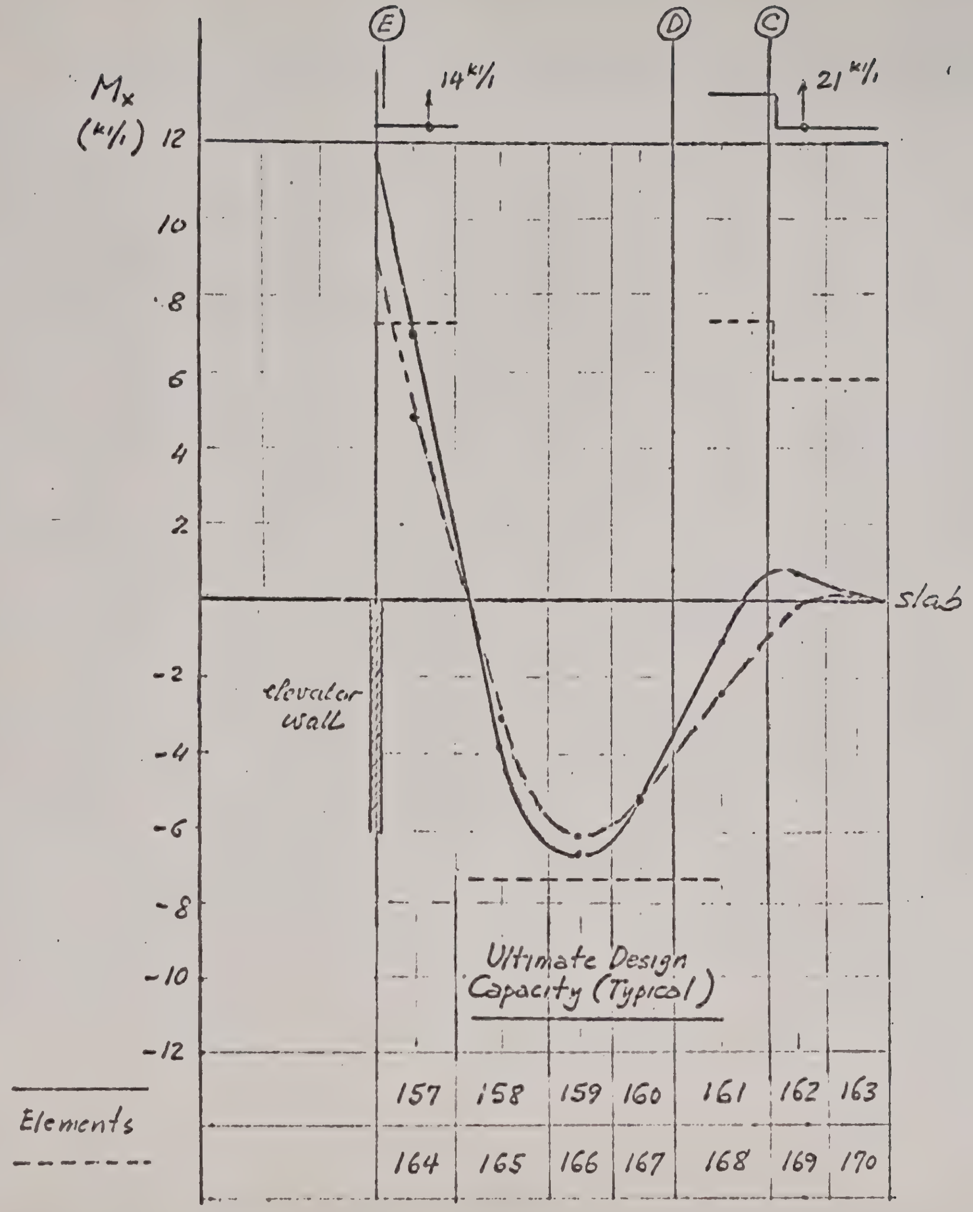
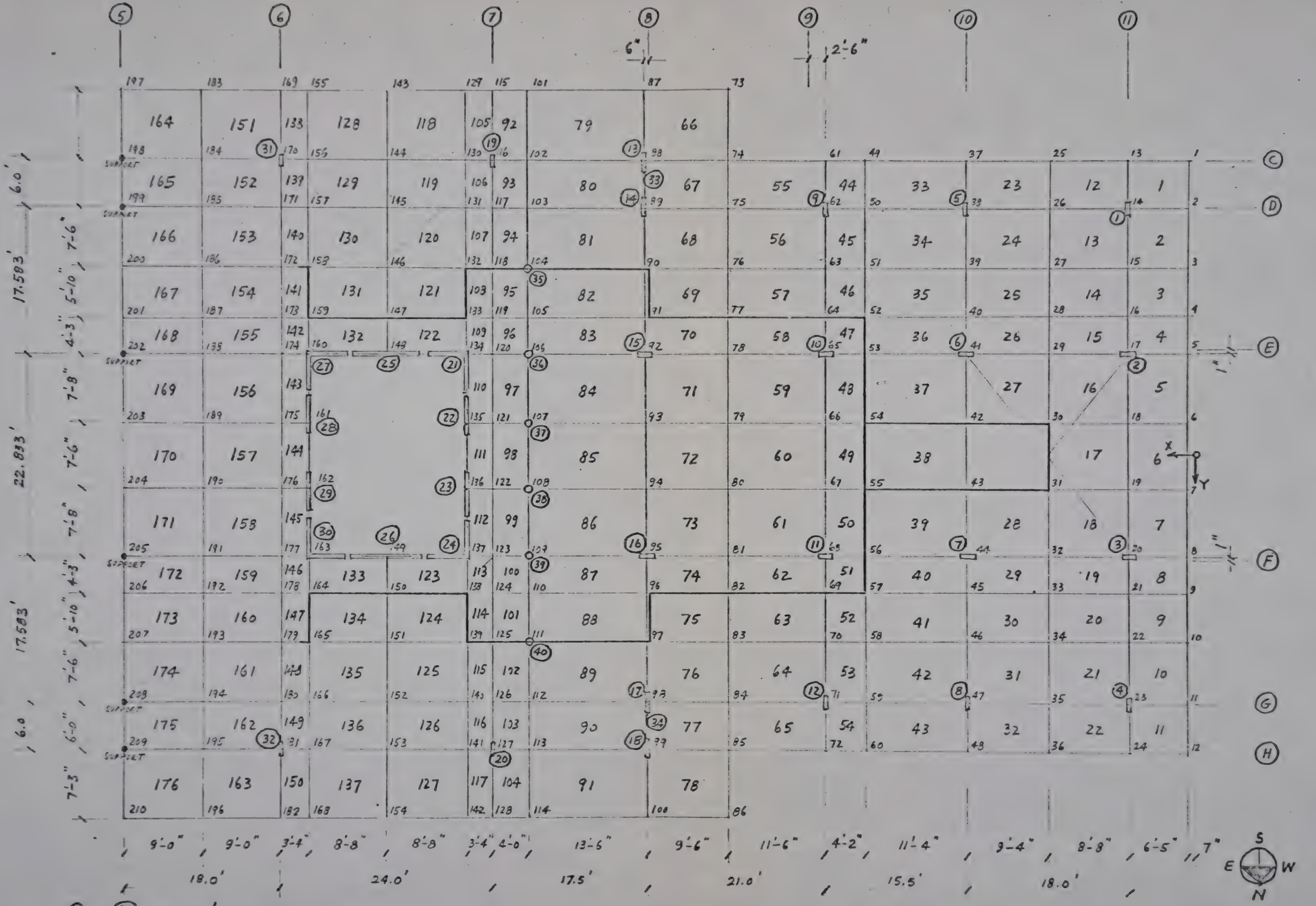


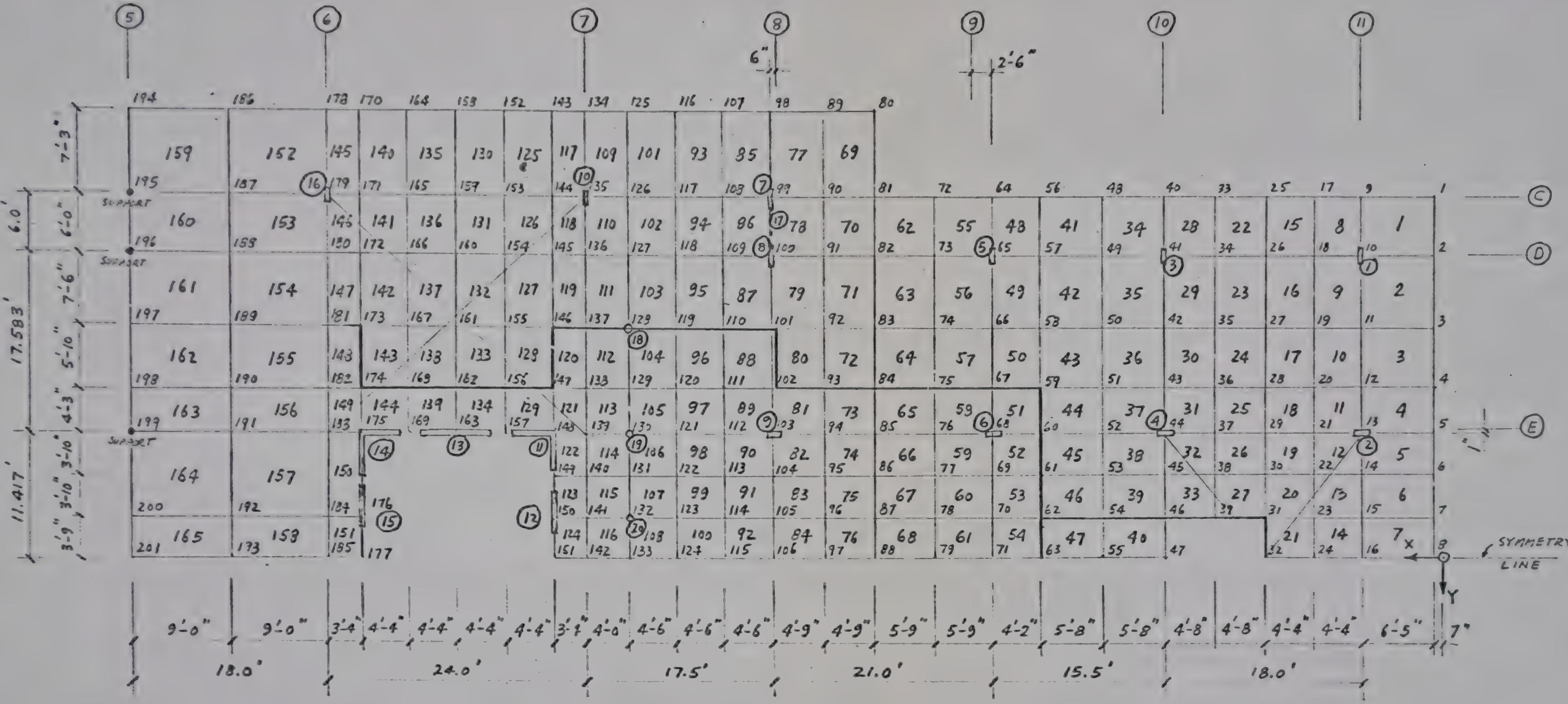
FIGURE 1.3g SLAB BENDING MOMENT DIAGRAMS FROM RUN 2B, FIGURE 1.2 GRID



① → ㉜ on grid denote columns from roof to 16th floor

㉟ → ㊵ denote shores

Figure 1.4 FINITE ELEMENT "COARSE" GRID FOR WEST SIDE ROOF ANALYSES



① → ⑯ on grid
denote columns from
roof to 16th floor

⑱, ⑲, ⑳ denote shores

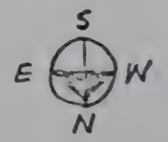
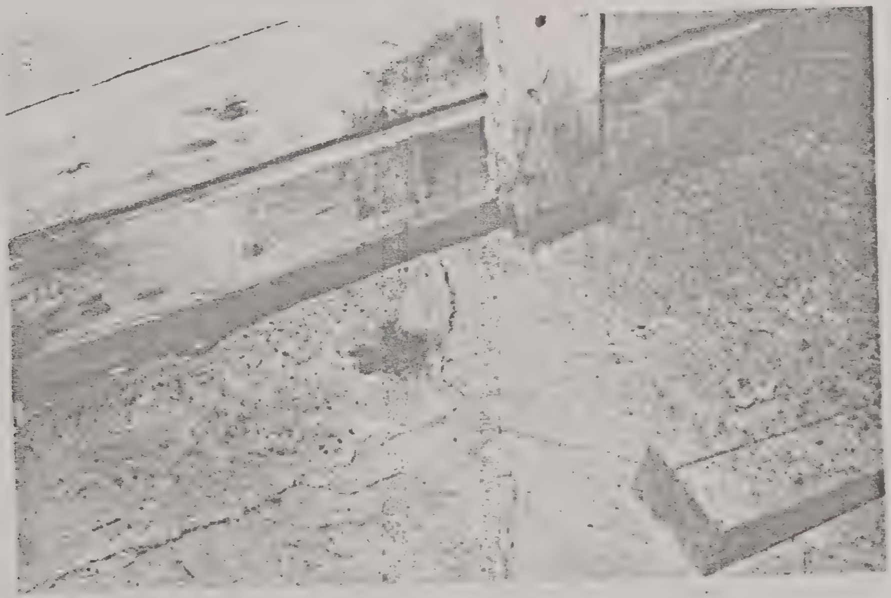
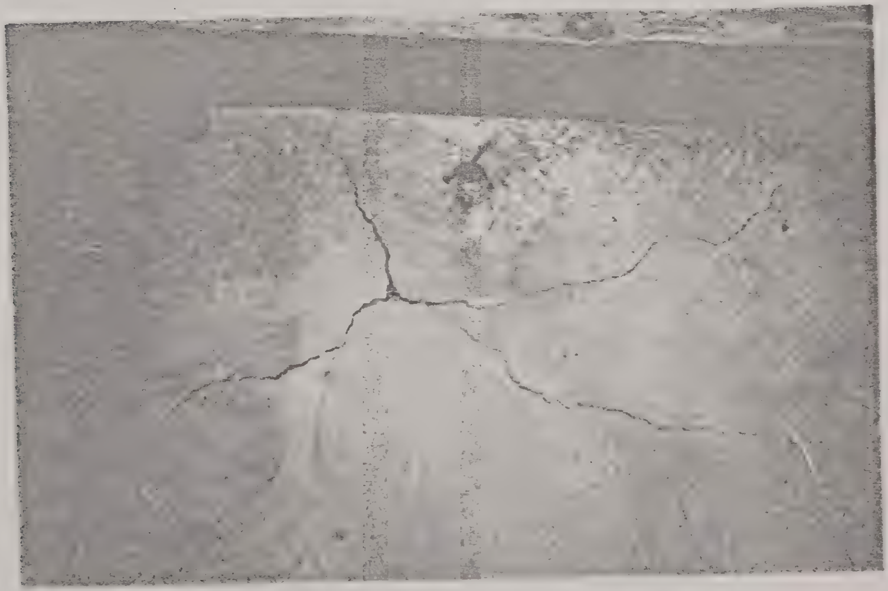


Figure 1.5 FINITE ELEMENT "FINE" GRID FOR WEST SIDE ROOF ANALYSIS

FIGURE 1.6 ROOF SLAB CRACKS OVER E8 AND F8



(a) Top Surface Over Column E8



(b) Top Surface Over Column F8

2.

GENERAL INVESTIGATION OF CONCRETE

The purpose of the investigation of concrete in the structure at 2000 Commonwealth Avenue was to determine the strength of the concrete, particularly at critical locations, at the time of collapse. To accomplish this purpose the investigation proceeded through several phases.

A. REVIEW OF RECORDS AVAILABLE FROM CONSTRUCTION

Design Documents and Boston Building Code

The concrete compressive strength for each portion of the structure is specified by the Structural Engineer on structural plans S1 and S6, which are included in Appendix II.4. The specified strengths, 3000, 4000, and 5000 psi, are noted to be the ultimate compressive strength at 28 days.

On the structural plans and in the specifications (Section 3A included in Appendix II.4) the Engineer also specified several additional characteristics which the structural concrete was to possess. These characteristics included the amount of entrained air, limits on the slump at placement, required admixtures, and prohibited admixtures.

Although suggested mix designs or guideline criteria for mix designs were not included in the specifications, the Engineer did require that the proposed mix

designs be certified by an approved inspection company to insure that they would be capable of providing for all specified characteristics. Section 3A of the specifications also contain many provisions regarding inspection, testing, placement, and curing of concrete.

In addition to the requirements imposed by the design documents, several sections of the Boston Building Code were applicable to concrete installed in the subject structure. The pertinent sections as extracted from the 1964 edition of the Code are included in Table 2.1. All of these sections contain minimum requirements and pertain to concrete strength, testing, inspection, and general construction procedures.

Mix Designs and Delivery Records

As stated in Part I, Section 3, no specific trial mixes were made for this project. Mixes were proposed by J. H. McNamara, Inc., the ready-mix Concrete Supplier, in a November 7, 1969, letter to Toulon Construction Company. J. H. McNamara furnished a copy of this letter to the Commission and it is reproduced in Table 2.2. The submitting of this letter is all that was done relative to the requirements regarding mix designs in the Boston Building Code and the designers' specifications. The irregularities and deficiencies pertaining to this matter are noted in Part I, Section 4C.

Although the mix designs proposed in the November 7, 1969, letter make no reference to the specific cement or aggregate to be incorporated in the concrete, testimony indicated that Atlas or Atlantic Type II cement and Plainville (Massachusetts) sand and gravel were used.

J. H. McNamara, Inc. also furnished the Commission copies of delivery slips and invoices for all concrete delivered to the construction site. In every case

the information contained on the invoices checked with that shown on the delivery slips. These data indicate that 2500, 3000, 4000, and 5000 psi concrete was delivered to the site. The volumes of these classes of concrete were 105.5, 4780.5, 3495.5, and 517.5 cubic yards, respectively, for a total delivered volume of 8899 cubic yards.

The delivery slips and invoices do not show a cement content for any of the 2500 psi concrete. The basis for the delivered 2500 psi concrete is not known; for the lowest strength called for on the structural plans is 3000 psi.

The records indicate a variety of cement contents for the 3000 psi concrete and one delivery record shows the incorporation of Type III cement. The cement content records are summarized below:

3000 psi Concrete

Cement Content (lbs./cu. yd.)	Volume (cu. yd.)
Not indicated	1071.5 (including 3.0 with Type III cement)
540	3475.0
580	185.0
611	10.0
640	39.0
	4780.5

The delivery slips for the 218 cubic yards of 3000 psi concrete placed in the roof slab on December 3, 1970, do not indicate cement content. The slips for 85 of the 95 cubic yards placed in the roof slab on December 9, 1970, indicate 540 pounds of cement per cubic yard and for the other 10 cubic yards the cement content is not indicated.

Of the 3495.5 cubic yards of 4000 psi concrete, the data show 2993.5 cubic yards with a cement content of 580 pounds per cubic yard (as proposed for 4000 psi (B) concrete in the letter of Table 2.2), 125.0 cubic yards with a cement content of 640 pounds per cubic yard (as proposed for 4000 psi (D) concrete), and 377.0 cubic yards with cement content not stated.

Of the 517.5 cubic yards of 5000 psi concrete, the data show 489.5 cubic yards with a cement content of 705 pounds per cubic yard and 28.0 cubic yards with cement content not stated.

The 8899 cubic yards of concrete were delivered on 132 different days between December 5, 1969, and January 25, 1971. In Tables II.2.1a and II.2.1b of Appendix II.2 each individual delivery record is tabulated.

While the records of concrete delivered are consistent with the proposed mix designs in a general sense, there are numerous discrepancies. These discrepancies are summarized in Part I, Section 4C.

Testing Laboratory Records of Inspection

All of the reports of the Hub Testing Laboratory were examined. These included those that were filed with the Boston Building Department and additional ones provided by Hub to the Commission as representing their complete file.

Laboratory reports include 63 plant inspection reports covering inspection of batching on 44 different days between January 27, 1970, and December 3, 1970; 54 concrete placement reports covering site inspection on 49 different days between January 27, 1970, and December 7, 1970; 125 reports covering test results of cylinders fabricated on 46 different days between January 6, 1970,

and December 3, 1970; and 26 reports noting that fabricated cylinders were picked up at the site and delivered to the laboratory on 24 different days between January 28, 1970, and November 25, 1970.

Table II.2.1a in Appendix II.2 indicates the correlation between the inspection and cylinder testing performed and the concrete delivery records. As extracted from the 54 concrete placement inspection reports, comments relating to the location of concrete placements in the structure and relating to other specific points are also tabulated in Table II.2.1a.

The Laboratory reports do not cover plant inspection on days when a total of 3043 cubic yards of concrete was delivered to the project. They do not cover inspection of concrete placement on days when a total of 2735 cubic yards of concrete was delivered and there are no cylinder test results for days when a total of 2467 cubic yards of concrete was delivered.

Table II.2.2 tabulates all results shown on the Testing Laboratory's cylinder test reports. These reported results are summarized in Part I, Section 4C where it is noted that the scatter in the results is surprisingly low.

B. INVESTIGATION OF CURING CONDITIONS

The 16th floor and main roof slabs each were cast on two different days, the 16th floor slab on November 18 and 20, 1970, and the main roof slab on December 3 and 9, 1970. Because the temperatures in Boston between those dates and the day of the collapse were below a range at which freshly deposited concrete most effectively gains its potential strength, it was necessary to investigate several factors relating to concrete curing conditions.

Requirements of Design Documents

The designers' specifications contain four pages (Division 3, Section 3A, pages 3A-6 through 3A-9) of provisions regarding protection measures to be taken for the warm moist curing of concrete placed in cold weather. These pages are included in Appendix II.4.

Testimony of Protection Provided

Testimony of witnesses before the Commission indicated that the only purposeful cold weather protection provided for the curing of concrete in the structure at 2000 Commonwealth Avenue occurred on one or two occasions in the winter of 1969-70. During November and December 1970 and January 1971, the only cold weather protection provided was such as may incidentally have occurred as a consequence of day time heat provided for the mortar of the bricklayers who may have been working in adjacent spaces. The 16th floor and main roof slabs were not covered or enclosed in any way after casting of the concrete.

Analysis of U. S. Weather Bureau Records

Official U. S. Weather Bureau records from December 1, 1969, to March 31, 1971, for Logan Airport, Boston, Massachusetts, are included in Appendix II.2, Table II.2.10. Inasmuch as wintertime inland temperatures in this region normally are lower than those existing at Logan Airport, the temperatures at 2000 Commonwealth Avenue in November and December 1970 and January 1971 likely were lower than those listed in the Logan Airport records.

Because the initial structural failure occurred in the roof slab, the available temperature records were carefully analyzed for the periods during which the

roof slab concrete would have gained the strength it had at the time of collapse. These periods ranged from December 3, 1970, to January 25, 1971, and from December 9, 1970, to January 25, 1971. These records show the following:

	Min. Temp. (°F)	Max. Temp. (°F)	Ave. Temp. (°F)
December 3, 1970	42	57	50
Period December 3, 1970, to January 25, 1971	2	57	26
December 9, 1970	21	36	29
Period December 9, 1970, to January 25, 1971	2	46	25

A daily log of minimum, maximum, and average temperature is shown in Figure 2.1. This log extends to March ~~28~~ 23, 1971, the last day on which concrete samples were taken at the construction site.

Laboratory and field tests of both roof slab concrete placements have not indicated that freezing of the concrete occurred before its initial set; however, concrete cured under these temperature conditions gains its strength at a much lower rate than under the normal or specified minimum curing temperature. Considerable test data have shown that, with a Type II cement, the 28-day strength of concrete cured at 25°F is approximately 50 percent of the 28-day strength of the same concrete cured at 70°F.

C. SAMPLING AND TESTING OF CONCRETE AFTER THE COLLAPSE

Core Tests From Pieces of Collapsed Structure

During the period January 26 to February 15, 1971, the John J. Duane Company

roof slab concrete would have gained the strength it had at the time of collapse. These periods ranged from December 3, 1970, to January 25, 1971, and from December 9, 1970, to January 25, 1971. These records show the following:

	Min. Temp. (°F)	Max. Temp. (°F)	Ave. Temp. (°F)
December 3, 1970	42	57	50
Period December 3, 1970, to January 25, 1971	2	57	26
December 9, 1970	21	36	29
Period December 9, 1970, to January 25, 1971	2	46	25

A daily log of minimum, maximum, and average temperature is shown in Figure 2.1. This log extends to March 23, 1971, the last day on which concrete samples were taken at the construction site.

Laboratory and field tests of both roof slab concrete placements have not indicated that freezing of the concrete occurred before its initial set; however, concrete cured under these temperature conditions gains its strength at a much lower rate than under the normal or specified minimum curing temperature. Considerable test data have shown that, with a Type II cement, the 28-day strength of concrete cured at 25°F is approximately 50 percent of the 28-day strength of the same concrete cured at 70°F.

C. SAMPLING AND TESTING OF CONCRETE AFTER THE COLLAPSE

Core Tests From Pieces of Collapsed Structure

During the period January 26 to February 15, 1971, the John J. Duane Company

removed most of the collapsed portion of the structure from the basement area. As was convenient, certain intact slab and column pieces were set aside for examination and testing. From several of these saved pieces a series of concrete cores was taken during the period February 6 to 12, 1971. The compressive strength test results from these cores are tabulated in Appendix II.2, Table II.2.5.

On March 23, 1971, a second series of cores was taken from pieces which had positively been identified as coming from the roof slab. The test results for this set of cores are included in Table II.2.5. The laboratory reports for both series of core tests follow Table II.2.5 in the Appendix.

Core Tests From Uncollapsed Structure

Twenty-four concrete cores were taken from various places in the uncollapsed portion of the structure. These cores were obtained by Superior Drilling Company for Weidemann, Brown, Inc., consultants for the owners of the property, and were delivered to the Thompson and Lichtner Company, Inc. laboratory for testing. The cores were taken on March 13, 15, and 18, 1971, immediately after the structure had been shored and it was deemed safe to proceed.

The compressive strength results from these 24 cores have been made available to the Commission and are tabulated in Appendix II.2, Table II.2.3. The accompanying laboratory reports follow Table II.2.3. The results are also shown with the core locations in Appendix II.2, Figures II.2.1a through II.2.1s.

Swiss Hammer Tests

Non-destructive Swiss hammer tests were made at various places in the uncollapsed structure and on various collapsed pieces in order to give indication

of concrete strengths. Tests were made on several days beginning February 9, 1971, but were not carried on continuously because of below freezing temperatures. The results are tabulated in Appendix II.2, Table II.2.4 and are shown in the appropriate plan location in Figures II.2.1a through II.2.1s.

Tests of Cylinders Found at the Site

Five 6 x 12 inch concrete cylinders were found on the ground at the site. Three of these were marked 11/24/70 (an inspection report indicates that 3000 psi concrete was placed in the lower basement on this day) and two were marked 12/3/70.

The cylinders were taken to the laboratory on January 26 and 27, 1971, were cured in the laboratory damp room for two weeks, and were tested on February 10, 1971. The test results, which are contained in Appendix II.2, Table II.2.8, indicate high, average, and low compressive strength values of 3700, 3110, and 2630 psi, respectively. These results generally indicate that the concrete represented by these cylinders was from a 3000 psi mix and the data have no other significance.

Cement Content Tests

Chemical analyses for mix constitution of concrete in certain parts of the uncollapsed structure and in certain collapsed pieces were made on the concrete in fifteen cores. The results of these analyses are contained in Appendix II.2, Table II.2.7, to which the laboratory reports are appended.

Eleven of the fifteen specimens were from roof slab concrete, six being from the December 3, 1970, placement and five being from the December 9, 1970, placement. The cement content determinations are summarized below.

pld

	Cement Content Indicated on Delivery Slips and Invoices (lbs./cu. yd.)	Cement Content from Chemical Analysis (lb./cu. yd.)		
		High	Average	Low
December 3, 1970 Roof Slab Concrete	Not Indicated	596	535	493
December 3, 1970 Roof Slab Concrete	540	530	501	488

The average cement content result is close to the value indicated in the proposed mix for the December 3 placement and is about 8 percent below that shown on the delivery slips for the December 9 placement.

In a given specimen of concrete a low value of cement content could be due to improper batching, segregation during placement, or both. Inasmuch as several different specimens were analyzed for each roof slab placement it is believed that the potential influence of segregation would be eliminated on the average. For the 3000 psi mix, a cement content of 501 pounds per cubic yard would adversely affect the strength gained at any given time by the concrete.

Cement contents found in single cores from the 16th, 14th, and 10th floor slabs were 573, 565, and 546 pounds per cubic yard, respectively. Each of these slabs was specified to be 3000 psi concrete and the cement content proposed in the mix design and shown on the delivery slip was 540 pounds per cubic yard. One core from the 3rd floor slab was found to have a cement content of 547 pounds per cubic yard. In this case the delivery slip showed 580 pounds of cement per cubic yard for the 4000 psi concrete specified for the 3rd floor.

D. STRENGTH EVALUATION

Roof Slab

The compressive strength results obtained from the December 3 and 9, 1970, placements are presented in Tables 2.3 and 2.4, respectively. Results obtained from cores which were cracked prior to test are not used in any strength evaluation. One core from π9 was steam cured prior to testing. This test was conducted to give a measure of the potential strength of the core and its result also has been excluded from evaluation of strength at the time of collapse.

Sound Core Test Results For
December 3, 1970, Placement

Age at Collapse 53 Days

Core Location	Age at Test (days)	Actual f'_c (psi)	
π15	67	2390	
π15	67	2840	
π15	67	2390	
			2540 ave.
Uncollapsed Roof Slab	102	1710	
	107	2580	
	107	2050	
	102	2240	
	102	2800	
	102	3010	
			2565 ave.

	Cement Content Indicated on Delivery Slips and Invoices (lbs./cu. yd.)	Cement Content from Chemical Analysis (lbs/cu. yd.)		
		High	Average	Low
December 3, 1970 Roof Slab Concrete	Not Indicated	596	535	493
December 9, 1970 Roof Slab Concrete	540	530	501	488

The average cement content result is close to the value indicated in the proposed mix for the December 3 placement and is about 8 percent below that shown on the delivery slips for the December 9 placement.

In a given specimen of concrete a low value of cement content could be due to improper batching, segregation during placement, or both. Inasmuch as several different specimens were analyzed for each roof slab placement it is believed that the potential influence of segregation would be eliminated on the average. For the 3000 psi mix, a cement content of 501 pounds per cubic yard would adversely affect the strength gained at any given time by the concrete.

Cement contents found in single cores from the 16th, 14th, and 10th floor slabs were 573, 565, and 546 pounds per cubic yard, respectively. Each of these slabs was specified to be 3000 psi concrete and the cement content proposed in the mix design and shown on the delivery slip was 540 pounds per cubic yard. One core from the 3rd floor slab was found to have a cement content of 547 pounds per cubic yard. In this case the delivery slip showed 580 pounds of cement per cubic yard for the 4000 psi concrete specified for the 3rd floor.

D. STRENGTH EVALUATION

Roof Slab

The compressive strength results obtained from the December 3 and 9, 1970, placements are presented in Tables 2.3 and 2.4, respectively. Results obtained from cores which were cracked prior to test are not used in any strength evaluation. One core from π9 was steam cured prior to testing. This test was conducted to give a measure of the potential strength of the core and its result also has been excluded from evaluation of strength at the time of collapse.

Sound Core Test Results For
December 3, 1970, Placement

Age at Collapse 53 Days

Core Location	Age at Test (days)	Actual f'_c (psi)	
π15	67	2390	
π15	67	2840	
π15	67	2390	
			2540 ave.
Uncollapsed Roof Slab	102	2710	
	107	2580	
	107	2050	
	102	2240	
	102	2800	
	102	3010	
			2565 ave.

The test results of cores taken from collapsed piece Π15 had about the same strength as that of the cores taken at a later time from the uncollapsed structure. This fact (corroborated by similar comparisons for other portions of the structure) indicated that sound cores taken from collapsed pieces could properly be used in evaluation of compressive strength at the time of collapse.

Figure 2.2 shows a typical relation between strength gain and age for a given Type II cement concrete curing under normal laboratory conditions at 70°F. Shown in the same figure are relations between strength gain and age for curing temperatures of 20, 30, 40, and 50°F. For any single mix design and certain specific moisture conditions curves somewhat different from those shown in Figure 2.2 may be obtained; however, most of the published data indicate that concrete cured at 25°F will attain only about 50% of its normal 28-day potential.

In every case an actual core test result gives indication of a higher compressive strength than would have existed for that concrete at the time of collapse. The reason for this is that, between the time of collapse and the time of test, the concrete in each core received additional curing. This additional curing included a certain number of days at the site plus two days of warm moist curing in the laboratory.

The interpretation of the core test results, in regard to evaluation of strength at the time of collapse, requires one to account for the rate of gain in the strength of the concrete achieved between collapse and test. This strength gain is a function principally of two parameters, age and temperature; however, the function also is influenced by factors such as the constitution of the concrete, the type of cement, and the moisture conditions during curing.

Several research programs have been conducted which focus on this issue. The results of these studies show very clearly that concrete which is air cured for 28 days at low temperatures will experience a rapid gain in strength if it is suddenly exposed to normal 70°F curing conditions; however, the results are all obtained with specific concrete mixtures and specific control of curing conditions. None of these reported results are directly applicable to the conditions at 2000 Commonwealth Avenue.

As may be seen in Figure 2.1, the temperatures at the site did increase, erratically, after the collapse. Whereas the average curing temperature of the roof slab concrete prior to collapse was about 25°F, the average temperature from the day of the collapse to the last day on which cores were taken was about 33°F. The evaluation of the reduction factor to be applied to actual core test results requires consideration of these erratically increasing site temperatures and the sudden change of the cores being taken to the laboratory for two days of warm moist curing at about 73°F.

Relating the results of published research and the experience of the Commission and its consultants to this particular problem, it is the Commission's opinion that the average compressive strength of concrete placed on December 3, 1970, was approximately 1900 psi at the time of collapse.

Sound Core Test Results For December 9, 1970, Placement

Age at Collapse 47 days

Core Location	Age at Test (days)	Actual f'_c (psi)	
π9	61	2100	
π9	61	1920	
π9	61	2090	
π20	62	1370	
π20	62	1570	
			1810 ave.
π9	106	2760	
π11	106	2970	
π20	106	2360	
			2695 ave.

On the same basis as described previously, it is the Commission's opinion that the average compressive strength of concrete placed on December 9, 1970, was approximately 1600 psi at the time of collapse.

Sixteenth Floor Slab

The compressive strength results obtained from the November 18 and 20, 1970, placements are presented in Tables 2.5 and 2.6, respectively. Excluding cores which were cracked prior to testing these data are summarized below:

Sound Core Test Results For 16th Floor Slab Concrete

Age at Collapse 66 or 68 Days

Core Location	Age at Test (days)	Actual f'_c (psi)	
(Nov. 18, 1970)			
Uncollapsed 16th Floor Slab	117	3710	
	117	3350	
	117	3440	
			3500 ave.
(Nov. 20, 1970)			
π71	84	3450	
π71	84	3400	
			3420 ave.

Based upon an evaluation of these results, including consideration of the temperature records from the dates of placement to the date of collapse, it is the Commission's opinion that the average compressive strength of the concrete in the 16th floor slab was approximately 3100 psi at the time of the collapse.

Other Portions of the Structure

Concrete in the 16th story columns was specified to have a compressive strength of 3000 psi at age 28 days. The columns probably were cast on December 1, 1970. One core taken from an uncollapsed 16th story column had a compressive strength on March 20, 1971, of 2780 psi. Two sound cores taken from a collapsed 16th story column (marked π19), when tested on February 12, 1971, had compressive strengths of 3110 and 2910 psi. An evaluation of these test results indicates that strength of 3000 psi had not been attained at age 28 days or at the time of the collapse.

The 15th floor slab was cast on November 6 and 9, 1970. Two cores from the uncollapsed portion of this slab were tested, one on March 15 and the other on March 17, 1971. The actual compressive strengths were 3560 and 3360 psi on those days. An evaluation of these results to account for the maturity at age 28 days indicates that the specified strength probably was not attained. This conclusion was reinforced by the results of Swiss hammer tests made at several locations on February 22, 1971. The average compressive strength indicated by the Swiss hammer tests at nine slab locations was 2540 psi.

The data from tested cores and Swiss hammer tests for slabs of the uncollapsed structure below the 15th floor, with values adjusted to strength at age 28 days and to strength at the time of collapse, indicate that the compressive strength was generally below that specified at age 28 days but generally above the 28-day specified value at the time of collapse.

Although not a factor in the collapse, subsequent core and Swiss hammer tests indicated that the concrete in the mechanical room (penthouse) floor slab did not attain its specified 3000 psi strength at age 28 days.

TABLE 2.1

EXTRACTED SECTIONS OF 1964 BOSTON BUILDING CODE WHICH PERTAIN TO CONTROLLED CONCRETE

Sect. 2603. Inspection of Concrete.— The commissioner shall require an applicant for a permit involving the structural use of concrete to have a competent inspector at all times on the work while such concrete is being proportioned, mixed and deposited.

Sect. 2604. Tests of Materials of Reinforced Concrete.— The commissioner shall have the right to require reasonable tests from time to time to determine whether the materials and methods in use are such as to produce reinforced concrete of the necessary quality. Copies of the reports of such tests shall be kept readily available by the commissioner for a period of two years after the completion of the structure.

Sect. 2612. Controlled Concrete.—(a) When the proportions are to be established by tests, the tests shall be made in advance of the beginning of construction using the materials proposed and consistencies suitable for the work and in accordance with the provisions of section twenty-six hundred and thirteen. The relation between the average twenty-eight-day strength of the concrete and the water-cement ratio shall be determined by such tests for a range of values including all of the strengths called for on the plans. The water-cement ratio determined for each quality of concrete to be used shall allow sufficient margin of strength to provide for the exigencies of field operations. In no case, however, shall concrete exposed to the weather contain less than five bags of cement per cubic yard of concrete or have a water-cement ratio in excess of six gallons per bag. No change or substitution shall be made in the materials being used on the work without additional tests to determine the new water-cement ratios to be used.

(b) During the progress of the work, a reasonable number of compression tests may be required by the commissioner, but at least one set of specimens shall be tested for each one hundred and fifty cubic yards of concrete of a given strength, and not less than one set of specimens of each strength of concrete for each day's operation. Such tests shall be made in accordance with the provisions of section twenty-six hundred and thirteen. If the average twenty-eight-day strength of the tested specimens for any portion of the structure falls below the strength called for on the plans, the commissioner shall have the right to require load tests as specified in Part 23 on the portions of the building affected, and to order a change in the mixture for the remaining portion of the structure.

(c) Controlled concrete shall be proportioned, mixed and placed under the supervision of an approved concrete control engineer.

TABLE 2.1 Continued

Sect. 2613. Field Tests of Concrete.—(a) Specimens for compression tests of concrete, when required, shall be made and stored in accordance with the Standard Method of Making and Storing Compression Test Specimens of Concrete in the Field, and tested in accordance with the Standard Method of Making Compression Tests of Concrete, of the American Society for Testing Materials; provided, that each set shall consist of at least three specimens which shall be stored under moist curing conditions at seventy degrees Fahrenheit, and no specimens need be stored on the structure.

(b) If tests disclose a consistent relation between the seven-day and the twenty-eight-day strength of the concrete, the tests required during the progress of concrete work may be made at seven days and the strength at twenty-eight days determined therefrom.

Sect. 2616. Concrete Forms and Equipment.—(a) Forms shall be substantial and sufficiently tight to prevent loss of mortar from the concrete.

(b) Before concrete is placed equipment for mixing and transporting the concrete and forms shall be clean; the spaces to be occupied by the concrete shall be free of snow, ice and debris.

Sect. 2618. Transporting Concrete.—(a) Concrete shall be handled from the mixer to the place of final deposit by methods which will prevent the separation or loss of the ingredients. Under no circumstances shall concrete that is partially hardened be deposited in the work.

(b) Concrete otherwise meeting the requirements of this code but mixed at a distance from the structure in which it is to be deposited may be used provided the time elapsed between addition of the cement to the aggregate and its deposit in the forms does not exceed one hour.

Sect. 2619. Placing Concrete.—(a) Concrete shall not be placed until the forms and reinforcement have been inspected and approved by the inspector required by section twenty-six hundred and three.

(b) When concreting is once started, it shall be carried on as a continuous operation until the placing of the section or panel is completed. The top surface shall be kept generally level and accumulations of water on the surface shall be promptly removed. Where construction joints are necessary, they shall be made in accordance with section twenty-six hundred and twenty-two.

(c) Concrete shall be thoroughly compacted by puddling with suitable tools during the operation of placing, and thoroughly worked around the reinforcement.

Sect. 2620. Depositing Concrete in Cold Weather.—When depositing concrete at freezing or near freezing temperature, the concrete shall be maintained at a temperature of not less than fifty degrees Fahrenheit, and not more than one hundred and twenty degrees. The temperature of the concrete shall be maintained at not less than fifty degrees for at least seventy-two hours after placing. When necessary, concrete materials shall be heated before mixing. Dependence shall not be placed on salt or other chemicals for the prevention of freezing.

Sect. 2621. Curing Concrete.—(a) Exposed surfaces of concrete shall be kept moist for a period of at least seven days after being deposited.

(b) Forms shall be removed in such a manner and with such precautions as to insure complete safety of the structure.

TABLE 2.2 COPY OF A LETTER FROM J. H. McNAMARA, INC. TO TOULON CONSTRUCTION CO. CONTAINING PROPOSED CONCRETE MIX DESIGNS

Toulon Construction
2000 Commonwealth Ave.
Boston, Mass.

Nov. 7, 1969

	Class A 3000 psi	B 4000 psi	C 5000 psi	D 4000 psi
Cement - lbs. -	540	580	705	640
Sand - lbs. -	1410	1380	1220	1330
Gravel - lbs. -	1860	1860	1860	1860
Water - gals.-	34.0	35.0	36.3	35.5
W/C Ratio -	5.92	5.7	4.84	5.26
Darex - ozs. -	1.0	1.0	2.0	1.5
Slump - Inches -	4.0	4.0	4.0	4.0

Above weights are given in the "oven dry state", they will be adjusted at the time of batching for moisture, yield & workability. The Class D, 4000 design mix is given in order to achieve 2000 psi at 3 days.

Sincerely yours,

TABLE 2.3

TEST RESULTS FOR ROOF SLAB CONCRETE PLACED ON DECEMBER 3, 1970

Location	Core Diamter (inches)	Date Sampled	Date Tested	Age at Test (days)	Density (lbs./cu.ft.)	Actual f'_c (psi)	Comments	Cement Content (lbs./cu.yd.)
D-10½	4	3/13/71	3/15/71	102	N.A.	2710	Sound Appearance, Tested core includes end of #5 reinforcing bar	537
F¾-10¾	4	3/18/71	3/20/71	107	146.5	2580	Sound Appearance	493
E½-9¾	4	3/18/71	3/20/71	107	146.5	2050	Sound Appearance	512
F½-9½	4	3/13/71	3/15/71	102	145.4	2240	Sound Appearance	514
E½-7¾	4	3/13/71	3/15/71	102	144.6	2800	Sound Appearance	556
D½-7¼	4	3/13/71	3/15/71	102	144.6	3010	Sound Appearance	596

TABLE 2.3 (continued)

TEST RESULTS FOR ROOF SLAB CONCRETE PLACED ON DECEMBER 3, 1970

Location	Core Diameter (inches)	Date Sampled	Date Tested	Age at Test (days)	Density (lbs./cu.ft.)	Actual f'_c (psi)	Comments	Cement Content (lbs./cu.yd.)
π 15	2	2/5/71	2/8/71	67	148.9	2390	Sound Appearance	
(D-6¼)	2	2/6/71	2/8/71	67	148.2	2840	Sound Appearance	
	2	2/6/71	2/8/71	67	148.4	2390	Sound Appearance	
	4	2/6/71	2/8/71	67	145.9	1880	Tested core cracked from top to bottom	
	4	3/23/71	3/25/71	112	145.8	2300	Tested core had hairline crack extending down 3"	

TABLE 2.4

TEST RESULTS FOR ROOF SLAB CONCRETE PLACED ON DECEMBER 9, 1970

Location	Core Diameter (inches)	Date Sampled	Date Tested	Age at Test (days)	Density (lbs./cu.ft.)	Actual f'_c (psi)	Comments	Cement Content (lbs./cu.yd.)
π 9	4	2/6/71	2/9/71	61	146.9	2100	Sound Appearance	
$(D\frac{3}{4}-2)$	4	2/6/71	2/8/71	61	146.4	1920	Sound Appearance	
	4	2/6/71	2/8/71	61	146.9	2090	Sound Appearance	506
	4	3/23/71	3/25/71	106	146.6	2760	Sound Appearance	490
	4	3/23/71	3/31/71	112	146.9	2660	Sound Appearance, Core Steam cured for 5 hours at pressure of 150 psi and temperature of 300°F	
π 11	4	3/23/71	3/25/71	106	147.6	2310	Tested core cracked from top to bottom	492
$(D\frac{1}{4}-2)$	4	3/23/71	3/25/71	106	N.A.	2970	Sound Appearance, Tested core contained two reinforcing bars	

TABLE 2.4 (continued)

TEST RESULTS FOR ROOF SLAB CONCRETE PLACED ON DECEMBER 9, 1970

Location	Core Diameter (inches)	Date Sampled	Date Tested	Age at Test (days)	Density (lbs./cu.ft.)	Actual f'_c (psi)	Comments	Cement Content (lbs./cu.yd.)
Π 20 (D-2$\frac{3}{4}$)	4	2/7/71	2/9/71	62	149.4	1370	Sound Appearance	488
	4	2/7/71	2/9/71	62	147.3	1570	Sound Appearance	
	4	2/7/71	2/9/71	62	146.3	1330	Tested core cracked through middle	
	4	3/23/71	3/25/71	106	146.8	2360	Sound Appearance	530
	4	3/23/71	3/25/71	106	145.8	1750	Tested core badly cracked up 3" from base	
	4	3/23/71	3/25/71	106	146.8	2050	Tested core cracked completely across	

TABLE 2.5

TEST RESULTS FOR CONCRETE CORES FROM THE NOVEMBER 18, 1970, 16TH FLOOR SLAB PLACEMENT

Location	Core Diameter (inches)	Date Sampled	Date Tested	Age at test (days)	Density (lbs./cu.ft.)	Actual f_c (psi)	Comments	Cement Content (lbs./cu.yd.)
D$\frac{1}{2}$-8$\frac{1}{2}$	4	3/13/71	3/25/71	117	147.1	3710	Sound appearance	573
E$\frac{3}{4}$-8$\frac{1}{2}$	4	3/13/71	3/15/71	117	146.3	3350	Sound appearance	
E$\frac{2}{3}$-7$\frac{1}{4}$	4	3/13/71	3/15/71	117	146.7	3440	Sound appearance	

TABLE 2.6

TEST RESULTS FOR CONCRETE CORES FROM THE NOVEMBER 20TH, 1970, 16TH FLOOR SLAB PLACEMENT

Location	Core Diameter (inches)	Date Sampled	Date Tested	Age at Test (days)	Density (lbs./cu.ft.)	Actual f'_c (psi)	Comments	Cement Content (lbs./cu.yd.)
π71	4	2/10/71	2/12/71	84	N.A.	2960	Core cracked prior to test and contained a piece of reinforcing steel	
π71	4	2/10/71	2/12/71	84	147.2	3170	Core cracked prior to test	
π71	4	2/10/71	2/12/71	84	146.6	3450	Sound appearance	
π71	4	2/10/71	2/12/71	84	N.A.	3350	Core cracked prior to test and contained a piece of reinforcing steel	
π71	4	2/10/71	2/12/71	84	147.0	3400	Sound appearance	

TEMPERATURES TAKEN AT LOGAN INTERNATIONAL AIRPORT, BOSTON, MASS.
LOCAL CLIMATOLOGICAL DATA
U.S. DEPARTMENT OF COMMERCE
NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION
ENVIRONMENTAL DATA SERVICE

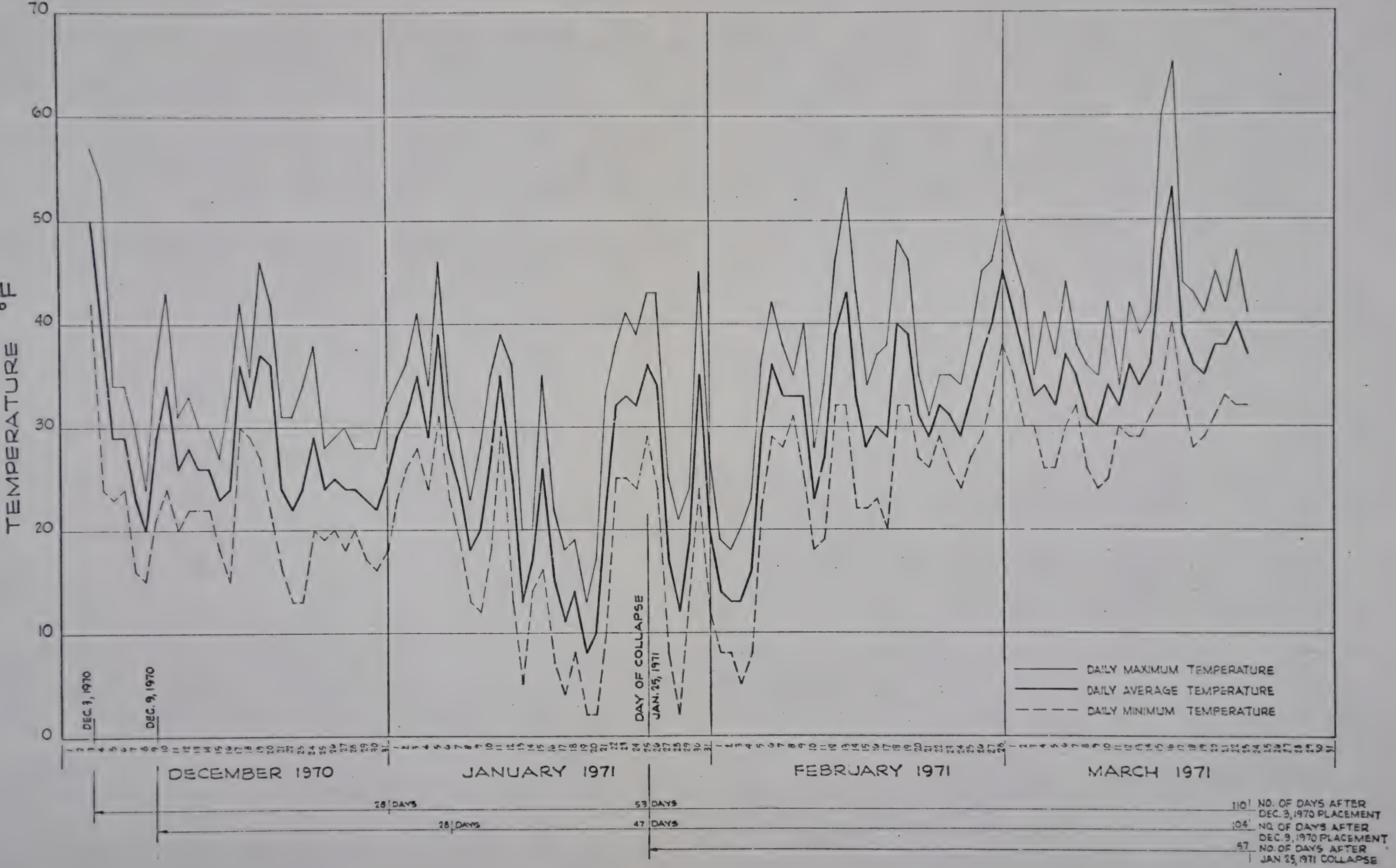


FIGURE 2.1
TEMPERATURE LOG
DECEMBER, 1970 THROUGH MARCH, 1971

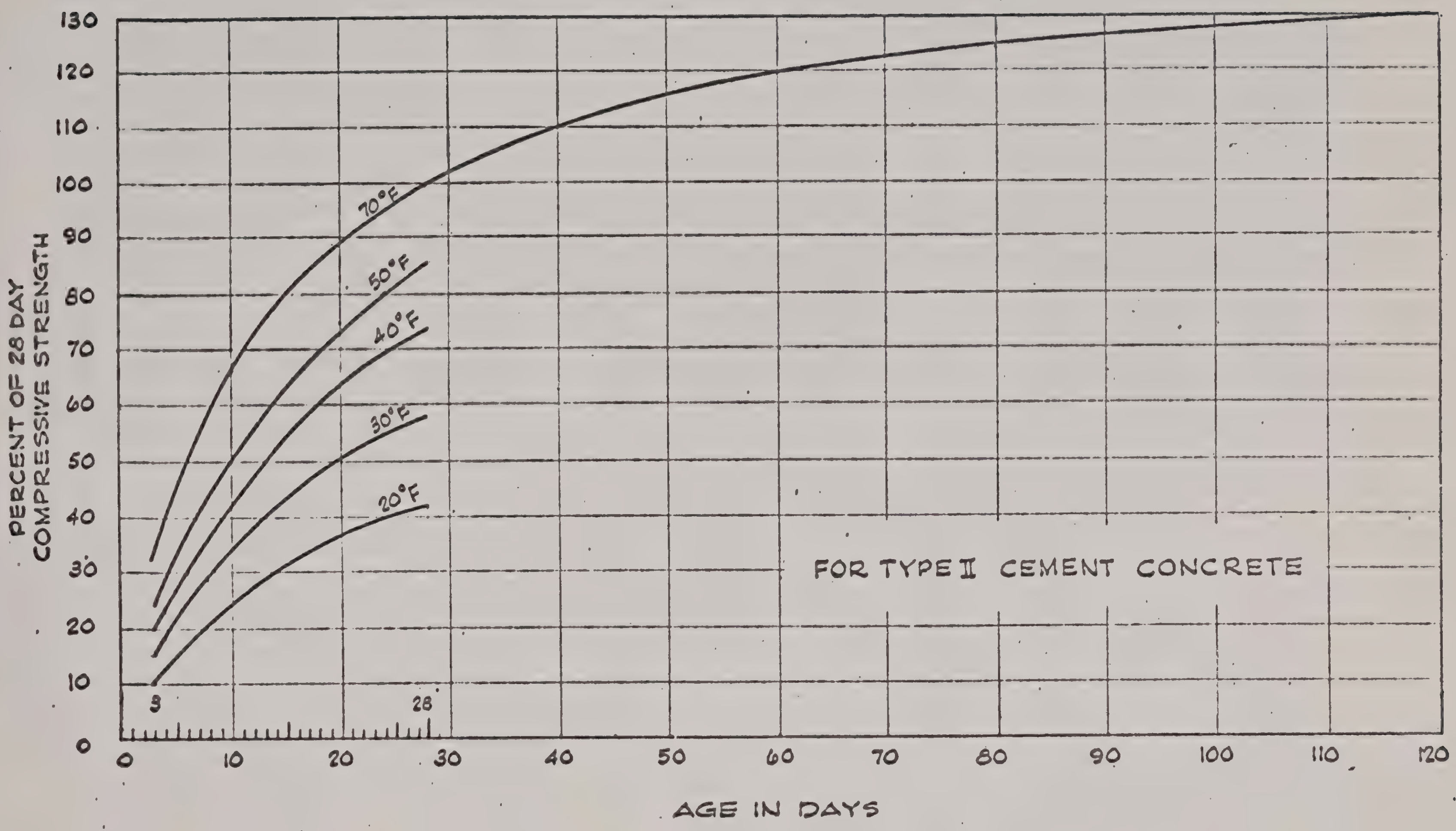


FIGURE 2.2
TYPICAL RELATIONSHIPS FOR RATE OF CONCRETE STRENGTH DEVELOPMENT

3.

GENERAL INVESTIGATION OF REINFORCING STEEL

A. THE SPECIFIED STEEL

The Structural Engineer proportioned reinforcement on the basis of the "ultimate strength" design method contained in the ACI 318-63 Code. This steel is identified on structural plans S1 to S9. Except for column ties, beam stirrups, and steel included in slabs on grade, which are specified to have a minimum yield stress of 40 ksi, all reinforcement is specified to have a minimum yield stress of 60 ksi and to meet the requirements of the ASTM A432 Specification for new billet steel.

On the basis of structural plans S1 to S9 the Structural Engineer later prepared bar lists (more than 100 sheets) which detailed every reinforcing bar in the structure. These lists contained bar size, bar length, bending details if any, an identification mark (e.g., D602, T1, MT101, C10, etc.) and the number of bars for each identification mark. To correlate the identification marks with the structural plans he also prepared a series of bar placing drawings. Each reinforcing bar list also contains reference to the required steel yield stress and ASTM A432 Specification requirements.

While it was not possible to make a detailed check of the specified reinforcement in the entire structure, a few inconsistencies were noted in the course of the investigation:

1. The east-west bottom slab steel in Bay E5-E6-F6-F5 and in Bay E7-E8-F8-F7 was detailed incorrectly for the typical floors and for the roof. To satisfy the ACI Code these Bars should have extended at least 6 inches into the elevator core walls. No such detail was contained on the structural plans. Had the bars been 21 feet 7 inches long instead of the 17 feet 6 inches specified, a detail drawing contained in the designers' specification would have satisfied the provision of the Code.

2. The ACI Code requires that at least 25 percent of the negative slab reinforcement in each column strip pass over the column within a distance "d" on either side of the column face. At column E4, E5, E8, E9, F4, F5 F8, and F9 in the roof slab, neither structural plan S-8 nor the bar placing drawings specifically provided for this requirement. A uniform distribution of the specified steel would nearly, but not quite, meet the requirement.

3. Based in part on the finite element analyses described in Part II, Section 1, it appears that the north-south top slab steel at the south face of the elevator core and the east-west top slab steel at the east and west faces of the elevator core is not sufficient to satisfy ACI Code requirements for the design conditions of the roof or typical floors. In the portions of the roof and typical floor slabs which are $7\frac{1}{2}$ inches thick on the east side, for example, 16 and 22 #4 bars, respectively are specified. This steel is not sufficient to resist design ultimate load bending moments resulting from finite element analyses.

   It must be noted, however, that the ACI Code permits a variety of techniques for determining design moments. Since the Structural Engineer has refused to furnish his calculations, his bending moment calculations for these particular areas are not available.

1. The east-west bottom slab steel in Bay E5-E6-F6-F5 and in Bay E7-E8-F8-F7 was detailed incorrectly for the typical floors and for the roof. To satisfy the ACI Code these bars should have extended at least 6 inches into the elevator core walls. No such detail was contained on the structural plans. Had the bars been 21 feet 7 inches long instead of the 17 feet 6 inches specified, a detail drawing contained in the designers' specification would have satisfied the provision of the Code.

2. The ACI Code requires that at least 25 percent of the negative slab reinforcement in each column strip pass over the column within a distance "d" on either side of the column face. At column E4, E5, E8, E9, F4, F5 F8, and F9 in the roof slab, neither structural plan S-8 nor the bar placing drawings specifically provided for this requirement. A uniform distribution of the specified steel would nearly, but not quite, meet the requirement.

3. Based in part on the finite element analyses described in Part II, Section 1, it appears that the north-south top slab steel at the south face of the elevator core and the east-west top slab steel at the east and west faces of the elevator core is not sufficient to satisfy ACI Code requirements for the design conditions of the roof or typical floors. In the portions of the roof and typical floor slabs which are 7½ inches thick on the east side, for example, 16 and 22 #4 bars, respectively are specified. This steel is not sufficient to resist design ultimate load bending moments resulting from finite element analyses.

   It must be noted, however, that the ACI Code permits a variety of techniques for determining design moments. Since the Structural Engineer has refused to furnish his calculations, his bending moment calculations for these particular areas are not available.

4. Although there is no mention of bent column dowels for the top of the top story columns on the structural plans, the structural engineer did include on the bar placing drawings a detail for them and the bar list (for the 9th and 16th story columns) includes some bent bars which may have been intended as bent column dowels. The bar list for the top story columns does not include enough bars for all top story columns and the bar placing drawings do not clarify exactly where the listed bars should be placed.

   If the inconsistencies noted above led to some confusion in the field, this confusion was amplified by the fact that the Steel Supplier did not schedule or supply the bent "dowels" contained in the Engineer's bar lists. The available evidence indicates that no bent dowels were placed to tie the top story columns to the roof slab.

5. For the detailed investigation of the roof slab collapse, a layout of the specified reinforcement for the south central portion of the roof was prepared. This layout is shown in Figure 3.1.

   In bay 6-7 two discrepancies between the Engineer's structural plans and his bar lists and bar placing drawings were noted. These are noted by A and B on the Figure.

B. THE SUPPLIED STEEL

Although the structural plans, specifications, and reinforcing steel bar lists clearly stated that all reinforcing steel should be new billet steel, a substantial percentage of the straight bars supplied for both columns and slabs was rail steel.

The Supplier has testified that he had only an oral contract with the Structural Subcontractor (but with cash payment by the Owner) to supply high-strength steel with no mention of billet or rail steel and that he had not seen the specifications or bid on the basis of meeting specification requirements. Nevertheless, the bar lists from which the Supplier filled the orders clearly noted that all steel should be new billet steel.

All rail steel found in the structure was manufactured by Northern Steel, the Steel Supplier. Sizes ranged from #4 up through #9. There is a preponderance of rail steel in the lower parts of the structure. Its use gradually diminished as the height of the structure increased and no rail steel was found in any piece identified as being from the 16th floor or main roof level.

In addition to the rail steel, billet steel manufactured in the United States (by Bethlehem Steel Corporation, Hawaiian Western Steel Limited, Milton Manufacturing Company, and United States Steel Corporation) as well as in Argentina and Taiwan was found in the structure. Physical test results for these several steels are described in Section 3C, following.

Although the supplied steel was not checked for every portion of the structure, the following irregularities or deficiencies were noted:

1. The designers' specifications require a series of tests for each 100 tons of reinforcing steel. No such tests were conducted.

2. As noted in Section 3 A.4 above, the Supplier did not schedule or deliver bent column "dowels" for the top of the top story columns.

3. As noted by C in Figure 3.1, the twelve #4 north-south top slab bars in bay 6-7 were not scheduled or supplied by the Supplier.

4. Reinforcement was supplied to the job in tagged bundles. Information on the tags included the number of pieces of a particular bar size, the bar length (for straight bars only), and the Supplier's bending designation mark (for bent bars only). In most cases the Supplier used the same bending designation as shown by the Engineer on the bar placing drawings and bar lists, but occasionally the Supplier designated bent bars with a mark which was used for other straight bars on the bar placing drawings and bar lists. One example of this situation is noted on Figure 3.1 where the Supplier gave bending marks for #4 bars at the south edge of the slab which were identical to marks given on the Engineer's placing drawings for top slab bars over column E5.

The Commission has attempted, unsuccessfully, to determine the procedures used by the Structural Subcontractor to select and place the reinforcement.

C. PHYSICAL TESTS

Structural plan S-1, dated February 1969, specifies that all reinforcing steel, except stirrups and ties and that included in slabs on grade, shall meet the requirements of the ASTM A432 Specification. This Specification was discontinued in 1968 and replaced by the Grade 60 part of ASTM A615-68 "Deformed Billet Steel for Concrete Reinforcement."

The ASTM A615-68 Specification contains certain provisions regarding the spacing, height, and gap for the bar deformations and lists minimum values of yield stress, ultimate tensile strength, and elongation at rupture. The latter values are:

f_y = 60 ksi minimum

f_u = 90 ksi minimum

Min. ult. elong. (8" gage) = 9% for #3, 4, 5, 6 bars
= 8% for #7, 8 bars
= 7% for #9, 10, 11, 14, 18 bars

Nine specimens of rail steel and fifteen specimens of billet steel were tested for deformation geometry and mechanical properties. The following list shows a few typical bar identification marks used by the several manufacturers.

Manufactured in United States

Northern Steel	N-6-RAIL
U.S. Steel Corp	G-3-N O-4-N-60
Bethlehem Steel Corp	B-4-N-60 B-6-N-60
Milton Manufacturing Co.	M-8-N (Grade 60) M-11-N (Grade 60)
Hawaiian Western Steel Ltd.	H-6-N-60

Manufactured in Argentina

AC-4-N-60

Manufactured in Taiwan

Tech-6-N-60

Table 3.1 summarizes the results of the tension tests. The actual test reports are contained in Table II.2.11 in Appendix II.2.

All of the nine rail steel specimens had strengths exceeding the specified minimum requirements of ASTM A615-68 Grade 60. For eight specimens ultimate elongation was obtained. While the ultimate elongations generally were significantly less than obtained from specimens of billet steel, only two specimens failed to meet the ASTM 615-68 Grade 60 minimum requirements for new billet steel.

Of the fifteen billet steel specimens, one had a very low ultimate elongation and another just failed to meet the yield strength requirements.

D. PLACEMENT OF STEEL

Several errors in the placement of the supplied steel were uncovered during the course of the investigation. Those which were felt to be significant are listed in Part I, Section 4D and they will not all be repeated here.

Slab Reinforcement at the East Face of the Elevator Core

The specified east-west top slab reinforcement for the roof slab at the elevator core is shown in Figure 3.1. Figure 3.2 shows the same reinforcement which was specified for the 3rd through 16th floor slabs. The Supplier's invoices indicate that these slab bars were shipped to the job site.

The photographs in Figure 3.3 show the fracture surfaces at the east face of the elevator core. These photographs were taken on February 2, 1971, after the hanging pieces to the north of the elevator core (from the 5th floor to the roof)

had been cut loose by the John J. Duane Wrecking Company. Photographs in Figure 8 of Part I and Figure 5.3 of Part II, taken on January 27, 1971, show the conditions as they existed after the collapse. The fractures at the roof slab, and at the 16th floor are shown in Figures 3.4 and 3.5, respectively. Although it would have been helpful to have made a closeup check of the fracture surfaces shows in Figures 3.4 and 3.5, it was not convenient to do so.

On March 16, 1971, a closeup investigation was made of fracture surfaces from the 8th to the 12th floors (this is as high as the available crane would permit). Figures 3.6 through 3.10 show sketches and photographs taken during this investigation. This investigation revealed numerous deviations in the number, size, and position of the top slab bars which were actually placed.

Combining the detailing error in the length of the east-west bottom slab bars with the errors and omissions in the placement of the east - west top slab steel, there was a definite plane of weakness at the east face of the elevator core. It is believed that the only #6 bars passing this vertical plane were those which protruded from some of the 8 x 22 inch beams inside the elevator core. The four specified #6 east - west top slab bars at each corner could not be found on any of the examined floors. The specified #4 top bars on each side of the hallway were present in some instances and not in others. In many locations the "top" steel was four to five inches from the top surface of the $7\frac{1}{2}$ inch slab. Finally, many of the hooked slab bars had only about 1 or 2 inches embedment into the elevator core wall.

Roof Slab Pieces π9, π11, π15, and π20

Pieces marked π9, π11, π15, and π20 were definitely identified as being from the collapsed portion of the roof slab. They are detailed in Appendix II.1 and

their locations are shown in Part II, Section 5, Figure 5.2. The reinforcement in them was carefully studied for conformity with the structural plans and bar placing drawings and the following items were noted:

Π9

1. At the east edge the specified north-south #4T and B are missing.
2. Parapet dowels are spaced at 15 and 16 inches instead of the specified 12 inches.
3. Instead of four east-west #4B there should have been six or seven #4B.

Π11

1. At the east edge the specified north-south #4T and B are missing.
2. Instead of fourteen north-south #4T there should have been eighteen #4T.
3. Instead of twenty-two north-south #4B there should have been only fifteen #4B.
4. Instead of nineteen east-west #4B there should have been no more than thirteen #4B.
5. The seven east-west #6 hooked bars passing over the column have a length of 14 feet 2 inches instead of the specified 16 feet 0 inches. Apparently, bars with mark D602 were used in place of bars with mark D601.
6. There are two stray east-west #6B hooked bars north of the column. These may have been placed in response to an erroneous designation on the bar placing drawings.

π15

1. The vertical positions of the two top layers are reversed from those specified on the drawings (i.e., instead of the north-south layer being the topmost layer, it lies under the east-west layer).
2. At the south side of the piece there should be at least thirteen east-west #6T instead of the eleven #6T which are present.

π20

1. At the south edge the specified east-west #4T and B are missing.
2. Parapet dowels are spaced at 14, 23, and 24 inches instead of the specified 12 inches.
3. From the south edge there should be twelve east-west #6T instead of the nine #6T and four #4T which were present. The four #4T were all bundled together.
4. At the north side of the piece there are six east-west #4T whereas no more than four #4T were called for.
5. At the east edge of the piece there are twenty-one east-west #4B whereas no more than fourteen #4B were called for. This overage is consistent with that noted in point 4 under π11.
6. At one place there is no concrete cover for the north-south #4T.

Column Ties

There was a consistent lack of column ties in the 30 to 40 inches above each floor level. Particularly, references to these omissions are noted in the sketches and photographs in Appendix II.1 for column pieces marked π1, π2, π7, π22, π27, π36, π46, π47, π48, π53, π56, π59, π72, π74, and π91. In piece π47 the first tie was 43 inches above the top of the floor slab. Although no attempt was made to relate these omissions to the large number of extra

tie bars which were found scattered around the ground at the south side of the building, about one thousand #3 bent tie (or stirrup) bars were still on the ground in various piles in May 1971.

No special attempt was made to document all of these cases.

Concrete Cover

Structural plan S-1 specifies concrete cover to reinforcing. These specifications are:

1. All concrete poured in forms but exposed to earth or weather:
    a. Main bars in beams and columns and bars larger than #5 in walls and slabs, 2 inches
    b. Bars #5 and smaller in walls and slabs, $1\frac{1}{2}$ inches
2. Concrete faces not exposed to earth or weather:
    a. Walls and slabs, 3/4 inch
    b. Beams and columns to stirrups or ties, $1\frac{1}{2}$ inches.

Placement of reinforcing steel, as regards concrete cover, was rather erratic. As an indication of incorrect placement, reference is made to Figures 3.6 and 3.9 where east-west top slab steel at the elevator core is shown. Whereas the concrete cover for this steel should have been $1\frac{1}{2}$ inches (3/4 inch cover to the north-south top steel plus 3/4 inch for a north-south #6 bar), it is seen that some such bars actually were placed below mid-depth of the slab. As indicated by Figure 3.11c the examples shown in Figures 3.6 and 3.9 are merely illustrative of the slab conditions which exist from place to place around the elevator core.

No special effort was made to note the concrete cover to reinforcement for each of the 87 collapsed pieces which are detailed in Appendix II.1. In a few instances, however, notations were made where concrete cover, larger or smaller than specified, was observed. Such errors are noted on the sketches of pieces π19, π20, π31, π53, π72, π79, π83, and π87. The photographs in Figures 3.11a and 3.11b show conditions in pieces π19 and π83, respectively.

Reinforcement Around Slab Openings

As discussed in Part II, Sections 5 and 6, the actual slab openings did not correspond to those which were used as the basis for the detailing and supplying of the slab reinforcement. The Commission has attempted, unsuccessfully, to determine how the Structural Subcontractor placed the supplied reinforcement, given that the actual openings did not correspond to those assumed in the detailing of the reinforcement.

None of the 87 pieces recovered from the collapse contained any of the large slab openings. Accordingly, it was not possible to determine how the Structural Subcontractor placed the supplied reinforcement at these particular locations.

TABLE 3.1 TENSION TEST RESULTS ON REINFORCING BARS

Mfg. Mark	Bar Size	Yield Stress (ksi)	Ult. Tensile Strength (ksi)	Ult. Elong. in 8 inches (%)
Rail Steel				
N-4-1	4	70.3	113.3	11.7
N-5-1	5	67.4	110.7	14.1
N-6-1	6	60.5	104.1	16.4
N-6-1	6	78.2	139.4	5.9 *
N-6-1	6	74.9	134.3	9.8
N-6-1	6	83.7	140.5	8.6 *
N-8-1	8	76.7	120.0	--
N-8-1	8	80.7	129.5	8.3
N-9-1	9	75.4	125.4	10.2
Billet Steel				
G-3-N	3	49.7	78.8	21.1
AC-4-N-60	4	62.2	109.2	12.5
B-4-N-60	4	68.2	100.2	17.2
B-4-N-60	4	73.1	117.5	12.1
B-4-N-60	4	72.5	109.5	16.4
O-4-N-60	4	64.7	97.1	18.0
B-6-N-60	6	67.6	114.0	15.6
H-6-N-60	6	71.9	122.7	3.5 *
Tech-6-N-60	6	61.3	99.0	15.6
Tech-6-N-60	6	59.8 *	95.9	17.2

TABLE 3.1 CONTINUED

Mfg. Mark	Bar Size	Yield Stress (ksi)	Ult. Tensile Strength (ksi)	Ult. Elong. in 8 inches (%)
Billet Steel (continued)				
M-8-N	8	61.1	102.0	13.7
B-8-N-60	8	63.6	102.8	15.6
M-11-N	11	66.1	100.4	16.4
M-11-N	11	69.4	101.9	18.8
M-11-N	11	68.3	104.3	15.6

Note: * indicates failure to meet ASTM A615-68 minimums (and, therefore, old ASTM A432 minimums.)

TABLE 3.1 TENSION TEST RESULTS ON REINFORCING BARS

Mfg. Mark	Bar Size	Yield Stress (ksi)	Ult. Tensile Strength (ksi)	Ult. Elong. in 8 inches (%)
Rail Steel				
N-4-I	4	70.3	113.3	11.7
N-5-I	5	67.4	110.7	14.1
N-6-I	6	60.5	104.1	16.4
N-6-I	6	78.2	139.4	5.9 *
N-6-I	6	74.9	134.3	9.8
N-6-I	6	83.7	140.5	8.6 *
N-8-I	8	76.7	120.0	--
N-8-I	8	80.7	129.5	8.3
N-9-I	9	75.4	125.4	10.2
Billet Steel				
G-3-N	3	49.7 **	78.8	21.1
AC-4-N-60	4	62.2	109.2	12.5
B-4-N-60	4	68.2	100.2	17.2
B-4-N-60	4	73.1	117.5	12.1
B-4-N-60	4	72.5	109.5	16.4
O-4-N-60	4	64.7	97.1	18.0
B-6-N-60	6	67.6	114.0	15.6
H-6-N-60	6	71.9	122.7	3.5 *
Tech-6-N-60	6	61.3	99.0	15.6
Tech-6-N-60	6	59.8 *	95.9	17.2

TABLE 3.1 CONTINUED

Mfg. Mark	Bar Size	Yield Stress (ksi)	Ult. Tensile Strength (ksi)	Ult. Elong. in 8 inches (%)
Billet Steel (continued)				
M-8-N	8	61.1	102.0	13.7
B-8-N-60	8	63.6	102.8	15.6
M-11-N	11	66.1	100.4	16.4
M-11-N	11	69.4	101.9	18.8
M-11-N	11	68.3	104.3	15.6

Note: * indicates failure to meet ASTM A615-68 minimums (and, therefore, old ASTM A432 minimums.)

** Design specifications call for 40 ksi minimum yield stress for #3 stirrup bars.

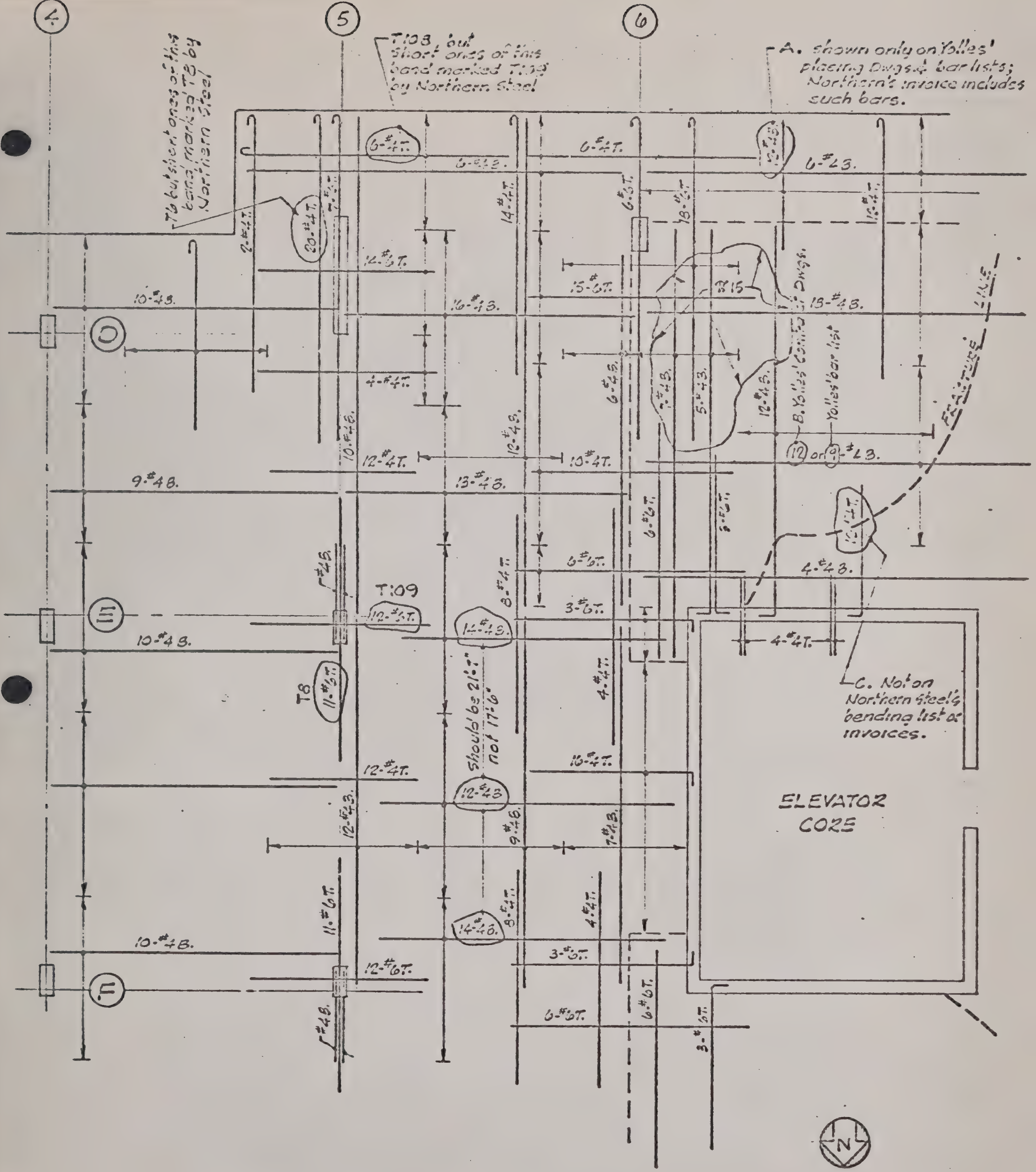


FIGURE 3.1 SCALE: 1/8"=1'-0"

SPECIFIED

REINFORCING LAYOUT FOR SOUTHCENTRAL PORTION OF ROOF SLAB

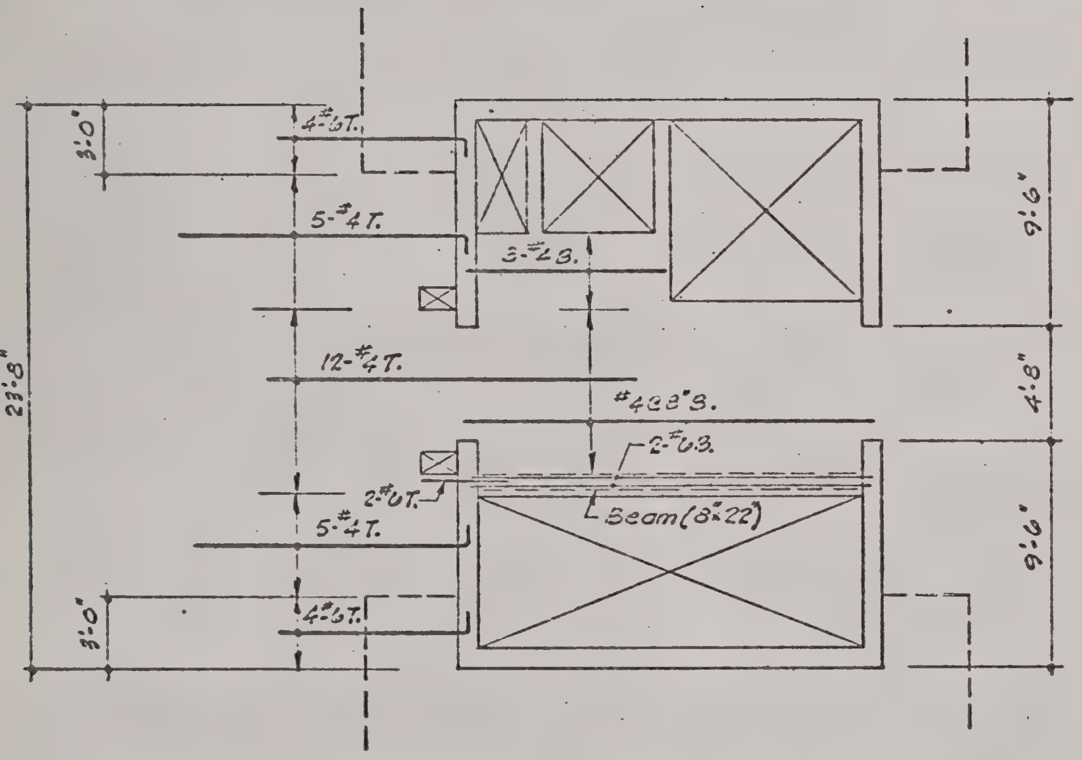


FIGURE 3.2 SCALE: 1/8"=1'-0"
SPECIFIED
EAST-WEST TOP SLAB STEEL AT
ELEVATOR CORE FOR 3rd TO 16th FLOORS

FIGURE 3.3 EAST FACE ELEVATOR CORE

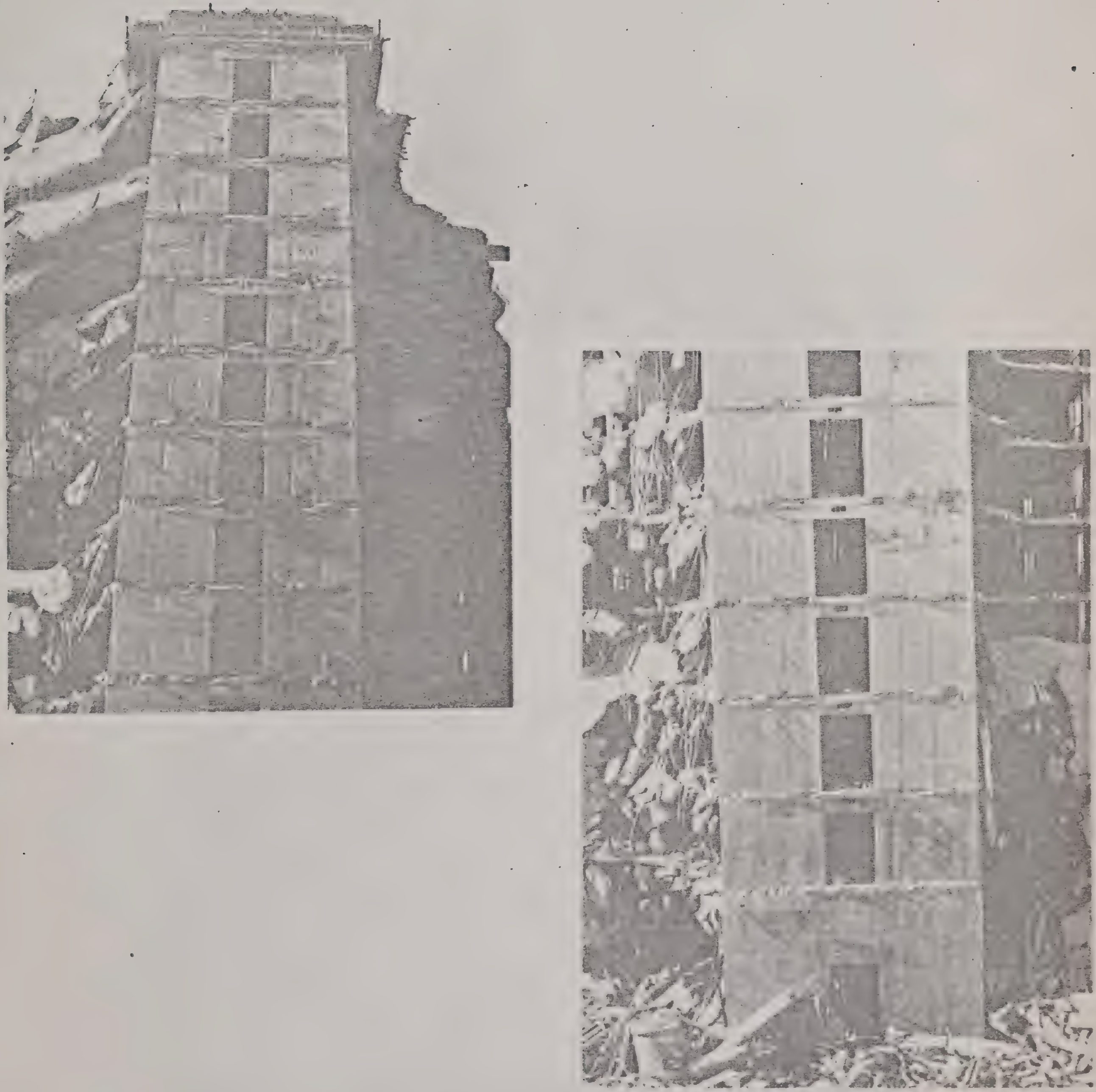
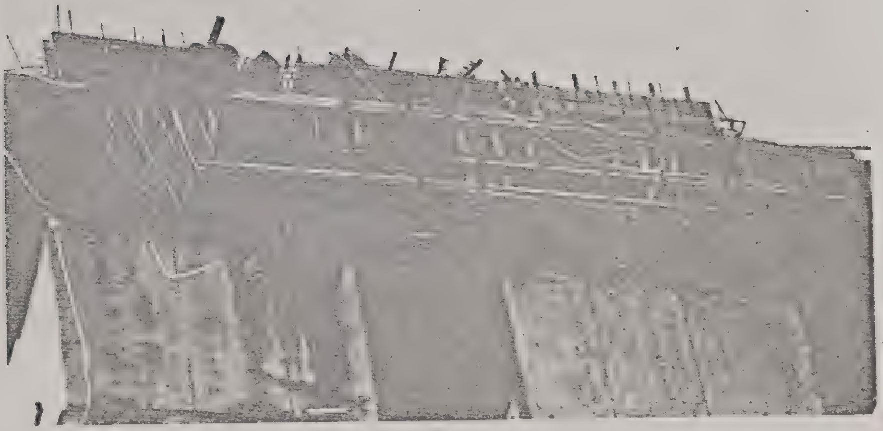


FIGURE 3.4 EAST FACE ELEVATOR CORE AT THE ROOF



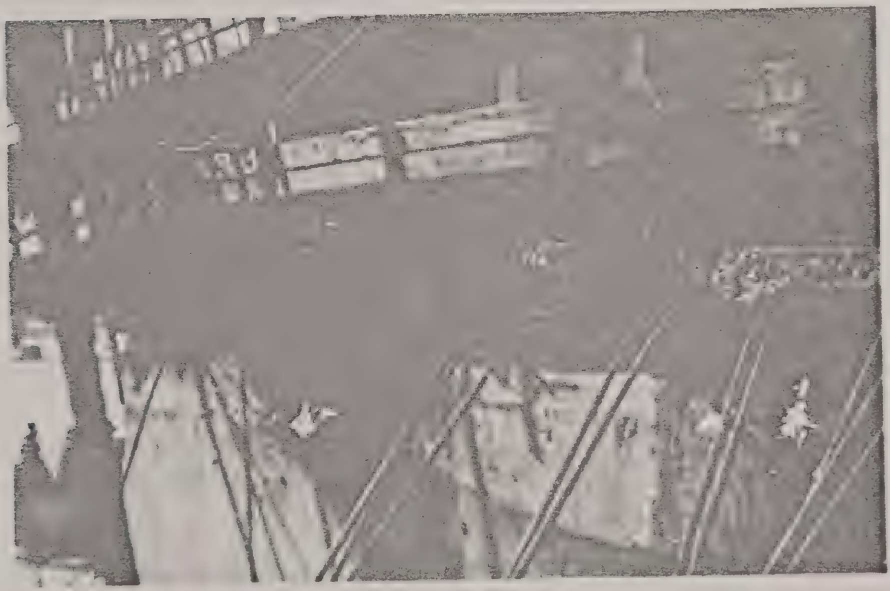
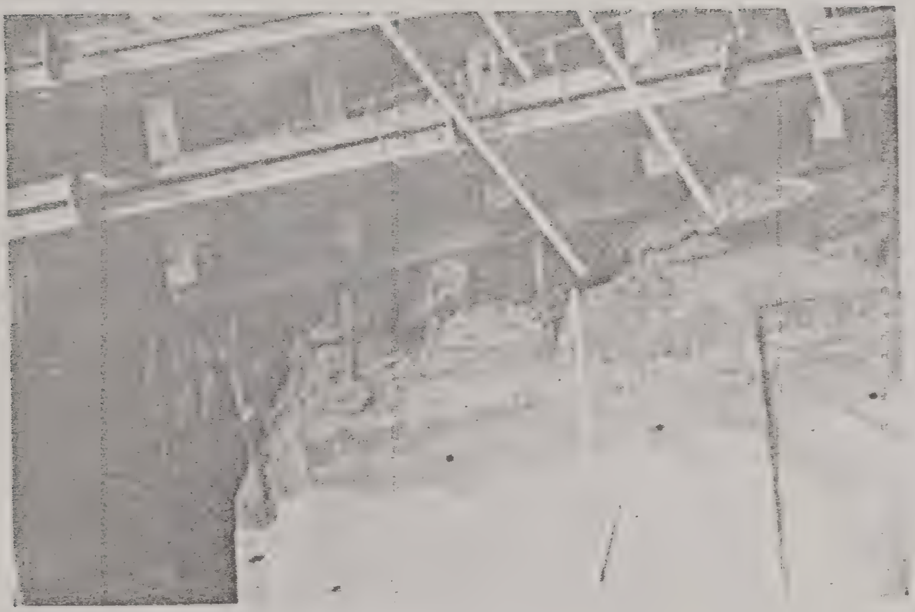


FIGURE 3.4 Continued



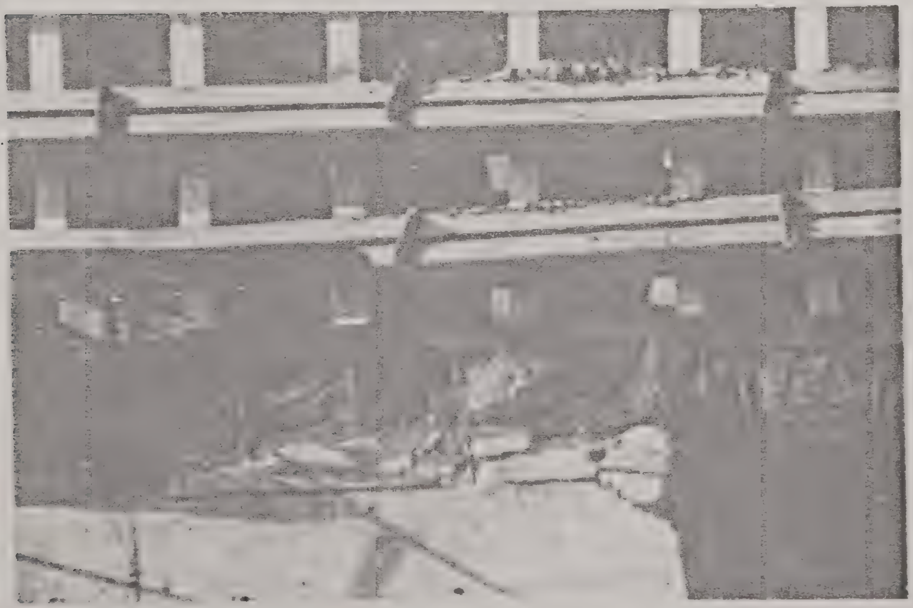
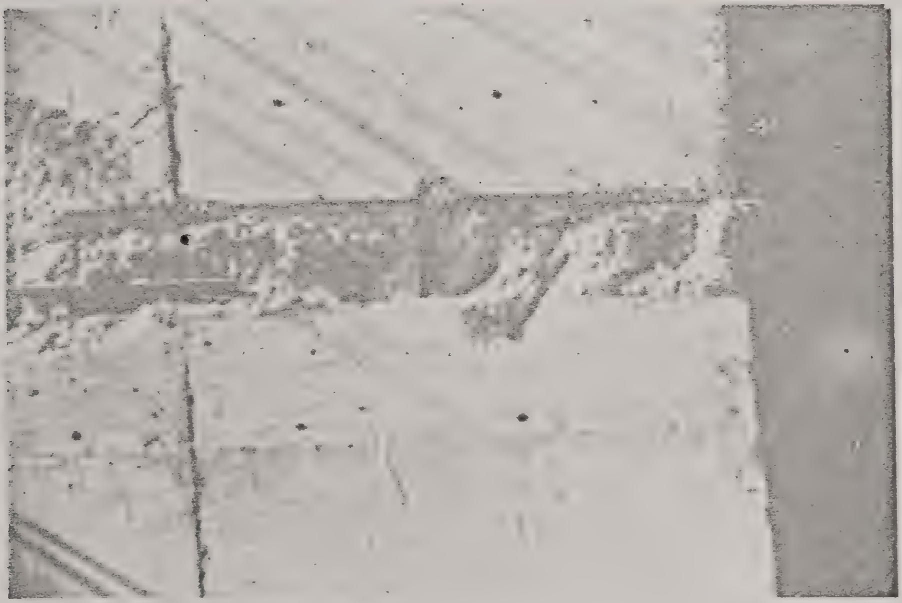
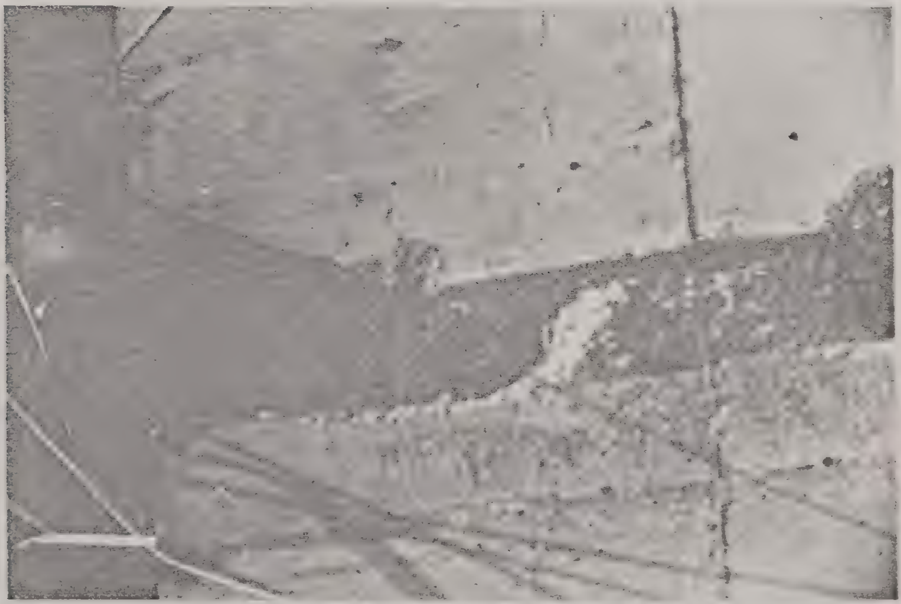


FIGURE 3.5 EAST FACE ELEVATOR CORE AT THE 16TH FLOOR



(a) Northeast Corner



(b) Southeast Corner

FIGURE 3.6 NORTHEAST CORNER OF THE EAST FACE OF THE ELEVATOR CORE AT THE 12TH FLOOR

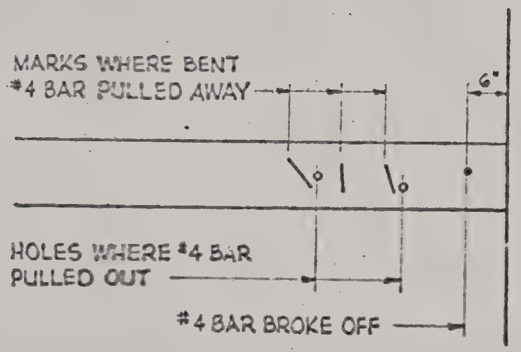
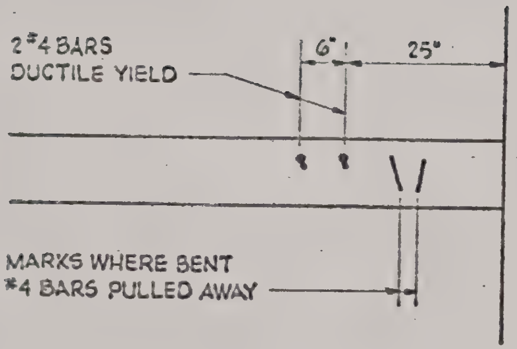


FIGURE 3.6 Continued

FIGURE 3.7 NORTHEAST CORNER OF THE EAST FACE OF THE ELEVATOR CORE AT THE 11TH FLOOR



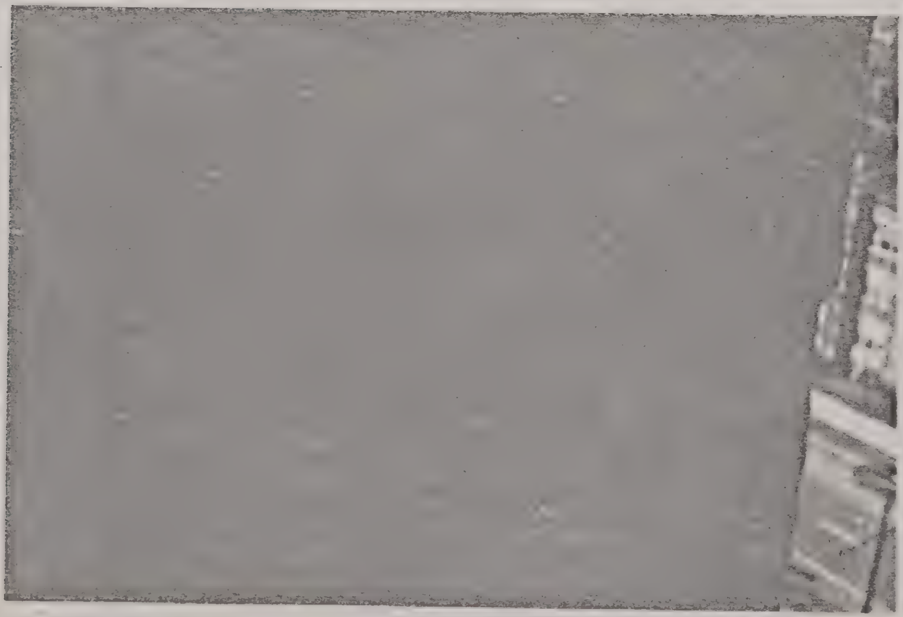


FIGURE 3.8 SOUTHEAST CORNER OF THE EAST FACE OF THE ELEVATOR CORE AT THE 10TH FLOOR

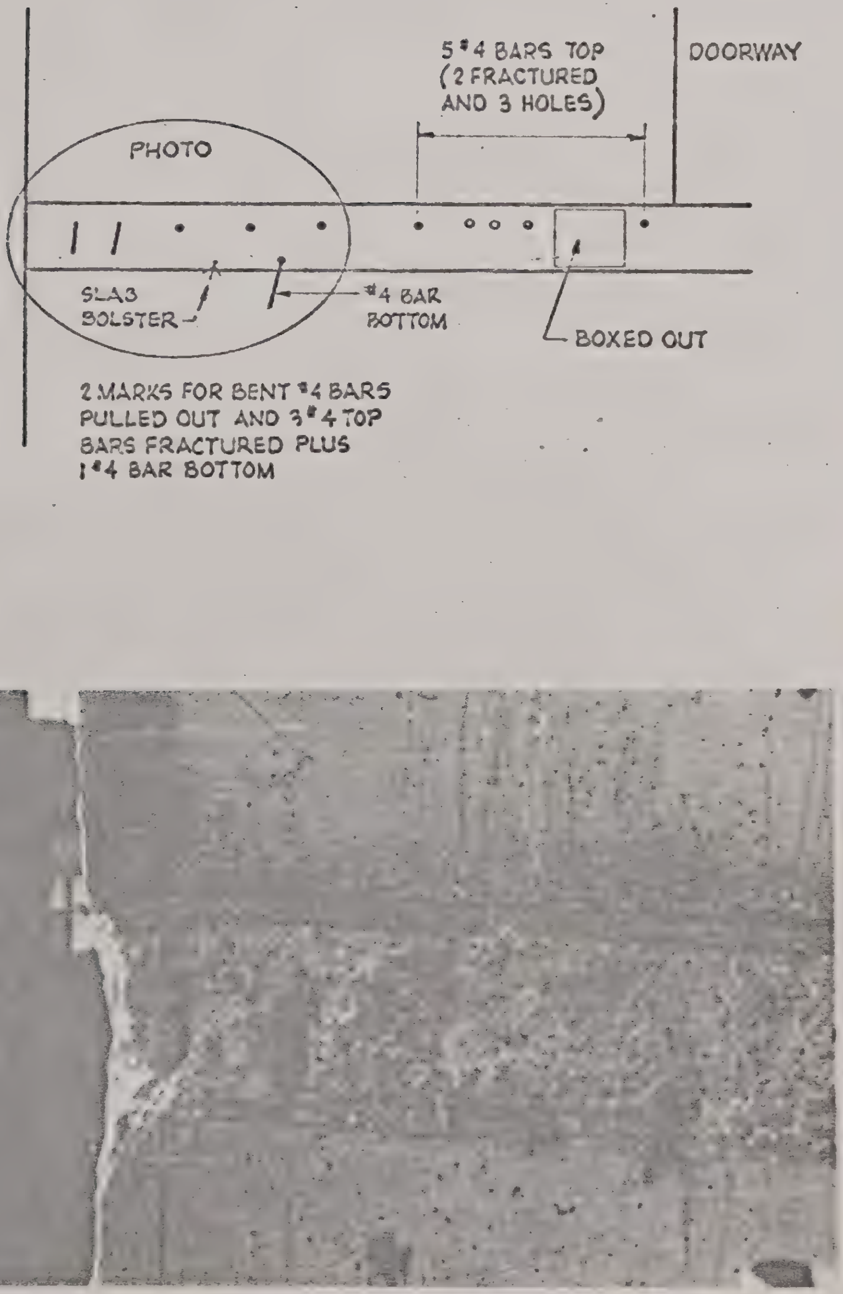
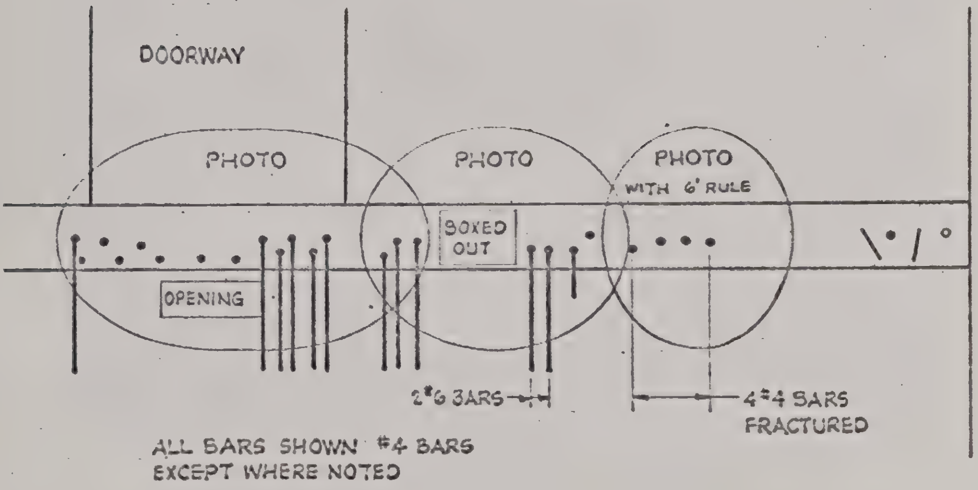


FIGURE 3.9 EAST FACE OF THE ELEVATOR CORE AT THE 9TH FLOOR



ALL BARS SHOWN #4 BARS EXCEPT WHERE NOTED

FIGURE 3.9 Continued

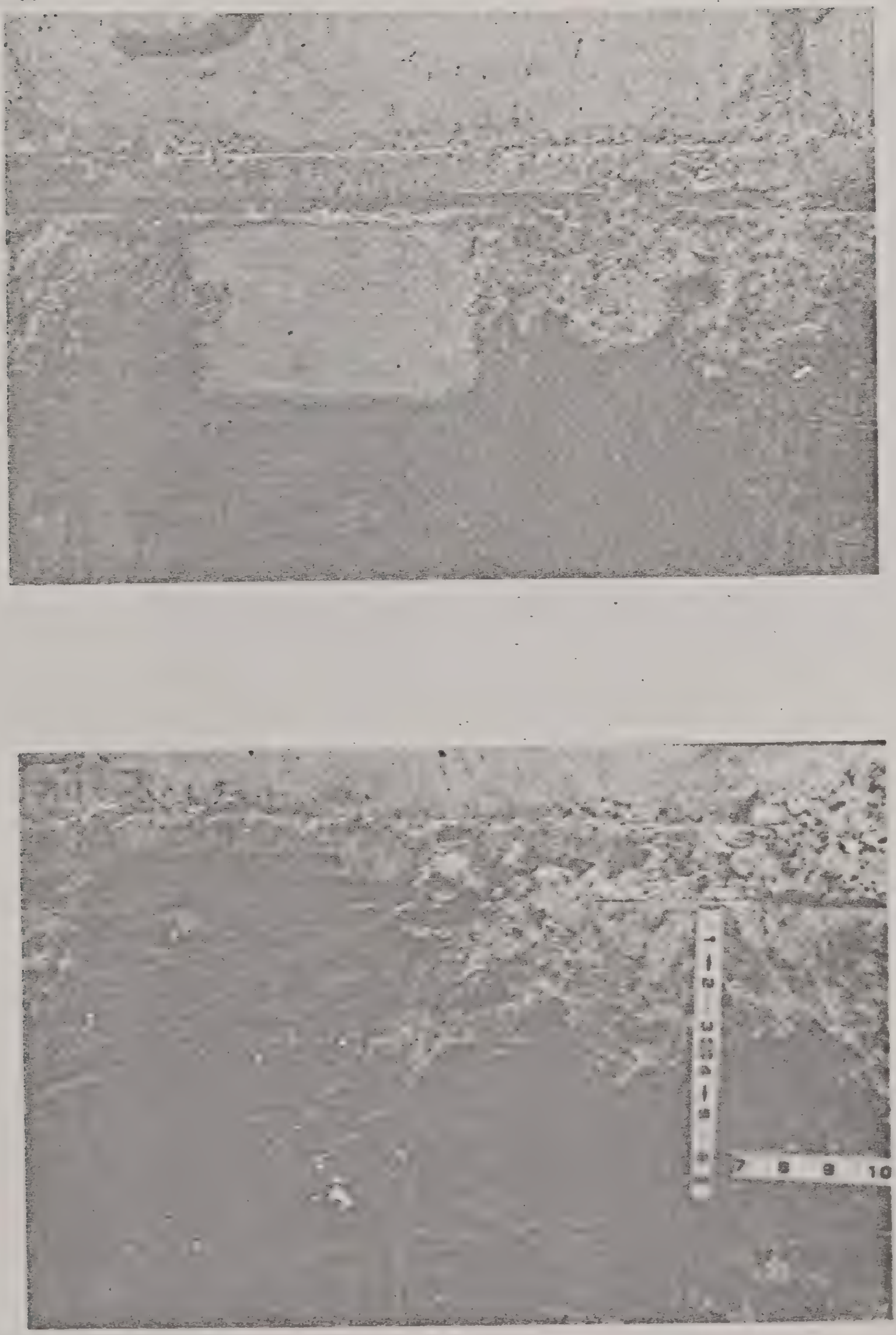
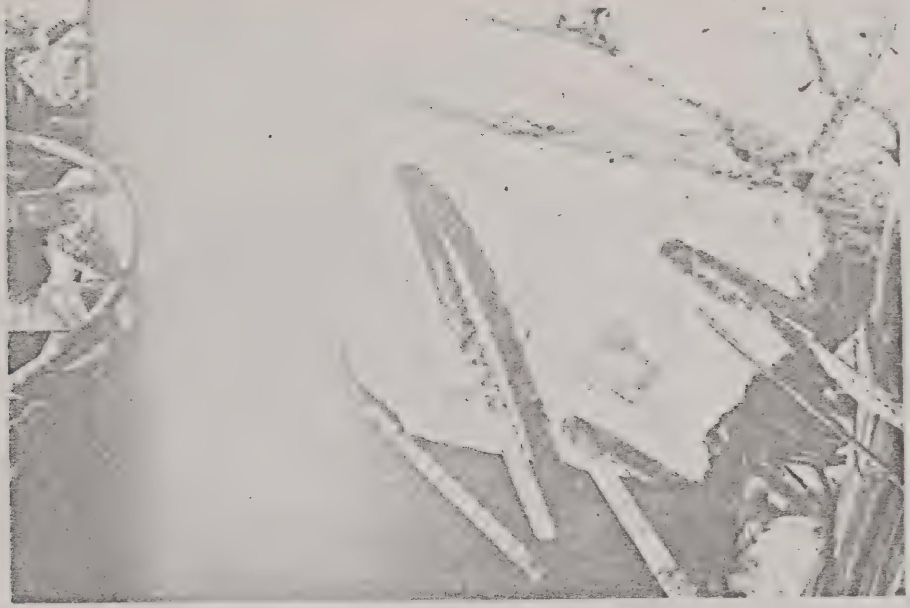


FIGURE 3.10 NORTHEAST CORNER OF THE EAST FACE OF THE ELEVATOR CORE AT THE 8TH FLOOR

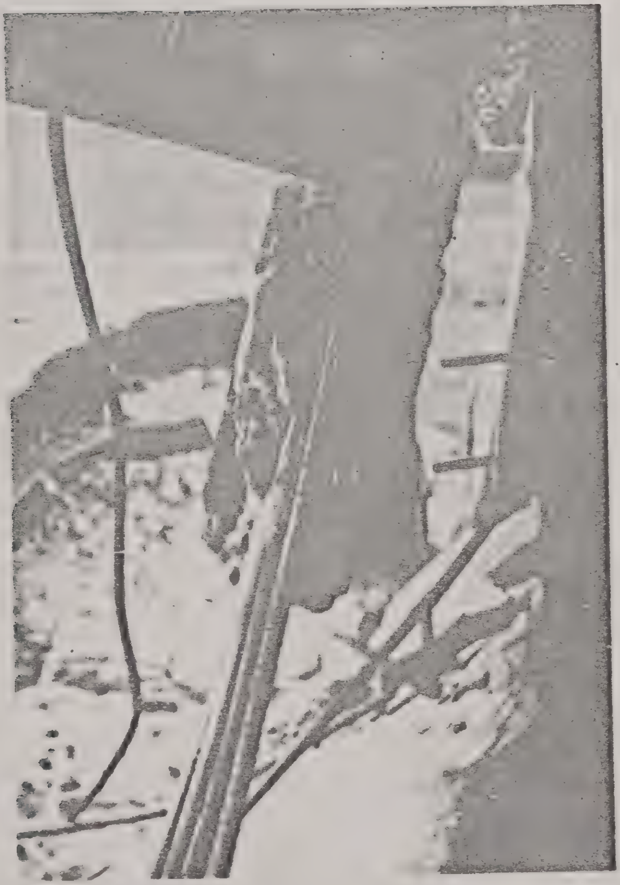
FIGURE OF CONCRETE COVER



(a) Piece T19 Top story column where concrete cover is 8" instead of the specified 1-7/8".

(b) P ypical floor slab piece showing p two #6T, one #4T, and one #4B

FIGURE 3.11 Continued



(c) Nine inch thick slab piece hanging from north face of elevator core. Note position of #6T bars. (Also note that these bars are rail steel.)

4.

GENERAL INVESTIGATION OF STRUCTURAL DESIGN

The structural design documents which were made available to the Commission are contained in Appendix II.4. These consist of nine plans, numbered S-1 through S-9 and dated February 9, 1969, and Division 3, Section 3A of the specifications, entitled "Plain and Reinforced Concrete." Copies of the plans were obtained from the City and, independently, from the Structural Subcontractor. The specifications were made available by the attorneys for the Structural Engineer.

During the course of the investigation a few inconsistencies between the structural plans and other design plans were noted. These related to slab openings and to dimensions of the mechanical room and are noted in Part II, Sections 5 and 6. Certain inconsistencies and deficiencies relating to the detailing of reinforcement by the Structural Engineer are noted in Part I, Sections 3 and 4D and in Part II, Section 3.

The purpose of this Section is to present the results of a review of the structural design. As explained in Part I, Section 4B, the review was conducted to determine conformity with the American Concrete Institute 318-63 Code. In basing his design on the ACI 318-63 Code the Structural Engineer elected to follow the "ultimate strength" design provisions contained therein. He chose to specify 60,000 psi minimum yield strength reinforcing steel for all but

secondary reinforcement and to specify either 3,000, 4,000 or 5,000 psi compressive strength concrete for various portions of the structure.

As noted in Part I, Section 4B the Structural Engineer's calculations were not made available. Thus the particular design options exercised and assumptions made by the Engineer are not known.

A. THE ROOF SLAB

The roof slab, as designed, does not support the mechanical floor wall beams once construction is completed. Rather, the wall beams are designed to transmit the loads from the mechanical floor (and the mechanical room roof) to the brackets which extend out from columns E4, E5, E8, E9, F4, F5, F8, F9, and the elevator core, whereupon these loads are then transferred into the designated columns and elevator core walls with little bending moment. The roof slab and mechanical floor slab between lines E and F would act as diaphragms (roof slab in compression and mechanical floor slab in tension) to prevent the brackets from rotating. Accordingly, only secondary bending moments would be transmitted into the designated columns and core walls. As construction never progressed to the point where this action would have taken effect, this condition was not examined in detail. As a general comment, this particular design concept, while somewhat unusual, appears to have been adequately treated.

Loadings .

The design gravity loadings for the roof slab include the dead weight plus 30 or 100 pounds per square foot live load. The design live load is 30 psf everywhere except for the area south of the south mechanical floor wall beam between lines 4-2/3 and 8-1/3 where it is 100 psf. These loadings conform to the requirements of the Boston Building Code.

Thickness

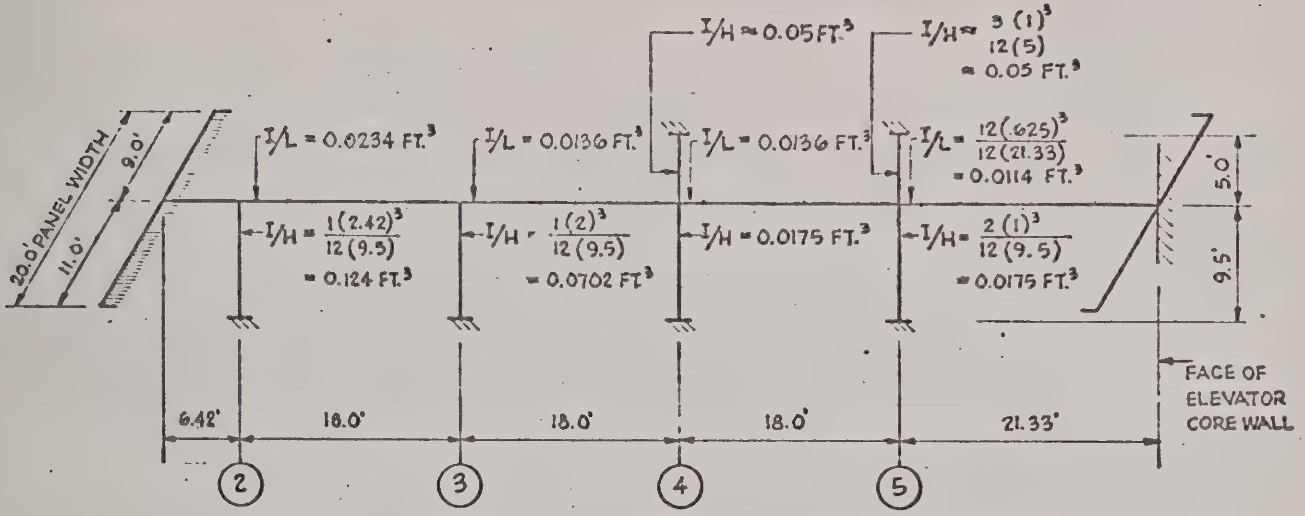
When design is based upon the "ultimate strength" method, Table 2101(e) of ACI 318-63 specifies certain minimum slab thicknesses. For slabs without drop panels and with reinforcement having a design steel yield stress of 60,000 psi, Table 2101(e) limits the minimum slab thickness to L/30. This minimum thickness requirement is intended to limit service live load slab deflections. According to this provision, the slab thickness at interior columns E4, E5, F4, and F5 should have been at least 8-3/4 inches. Similarly, around columns E8, E9, F8, and F9 the slab thickness should have been at least 9-1/4 inches. In all of these areas the thickness specified on structural plan S-8 is 7-1/2 inches. The 7-1/2 inches slab thickness would conform to the requirements of ACI 318-63, Table 2101(e) if the minimum steel yield stress used in proportioning the slab reinforcement were 40,000 psi instead of 60,000 psi. For this reason it is of interest to review the slab reinforcement specified on structural plan S-8.

Slab Reinforcement

It is assumed that the determination of slab design moments was based upon elastic frame analyses as provided for in ACI 318-63, Section 2103 (design according to the alternative method described in Section 2104 would not conform because the roof slab openings indicated on structural plan S-8 do not conform to Section 2104 requirements).

Calculated roof slab moments will differ modestly depending upon whether or not it is assumed that construction is completed up through the mechanical room (penthouse) floor slab before formwork for the roof slab is stripped. These differences will depend upon whether or not the slab is restrained by the brackets and walls which extend up to the mechanical room. In the frame analyses made in the review, restraint of these brackets and walls was assumed to be present for all loads.

Frame Analysis Along East-West Line E Columns


I/H ≈ 0.05 FT.³
I/H ≈ 3(1)³ / 12(5) ≈ 0.05 FT.³
20.0' PANEL WIDTH
9.0'
11.0'
I/L = 0.0234 FT.³
I/L = 0.0136 FT.³
I/L = 0.0136 FT.³
I/L = 12(.625)³ / 12(21.33) = 0.0114 FT.³
I/H = 1(2.42)³ / 12(9.5) = 0.124 FT.³
I/H = 1(2)³ / 12(9.5) = 0.0702 FT.³
I/H = 0.0175 FT.³
I/H = 2(1)³ / 12(9.5) = 0.0175 FT.³
5.0'
9.5'
FACE OF ELEVATOR CORE WALL
6.42'
18.0'
18.0'
18.0'
21.33'
2
3
4
5

$$F.E.M._{5\text{-ELEV.}} = \frac{1.5(0.094)(20)(21.33)^2}{12} + \frac{1.8[0.100(4)+0.030(16)](21.33)^2}{12} \times (0.75)^*$$

$$= 106.9 + 45.0$$

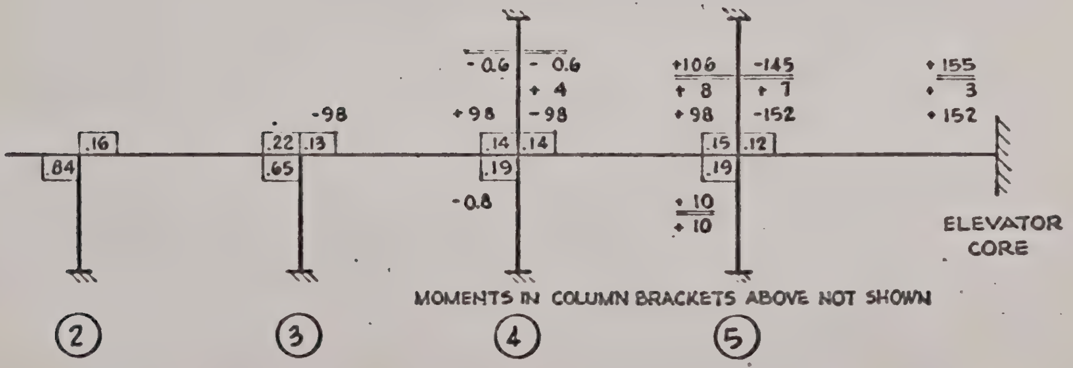
$$= 151.9 \text{ KIP FT}$$

$$F.E.M._{5\text{-}4} = \frac{1.5(0.094)(20)(18)^2}{12} - \frac{1.8(0.030)(20)(18)^2}{12} \times (0.75)^*$$

$$= 76.1 + 21.9$$

$$= 98.0 \text{ KIP FT}$$

* LIVE LOAD REDUCTION FACTOR



The Boston Building Code in effect at the time the design was completed does not permit any reduction in roof live loads. The live load reduction factor incorporated in the moment calculations would be permitted by the Boston Code only if it is assumed that the "roof" slab bounded by the mechanical room wall beams is not considered to be a roof slab.

Since the slab does not fall within all the limits set by Section 2104, the ACI Code does not permit the reinforcement at a "critical section" tc be proportioned for less than the calculated moments.

The bending moment of 155 kip feet is calculated at the face of the elevator core wall. The code requires that the top slab reinforcement at this location be proportioned for the moment existing at the "critical section," which in this case is 3-3/4 inches outside the face of the wall. The calculated bending moment 3-3/4 inches away from the face of the wall is about 150 kip feet.

It is noted that 11.83 feet of the total 20 feet panel width abuts the elevator core. Accordingly, it would be appropriate to provide 11.83/20 x 150 = 89 kip feet of resisting moment in steel entering the elevator core. The design provides for 3 #6 and 8 #4 east-west top bars entering the elevator core.

The permissible ultimate bending moment capacity for these bars is:

$$3\#6 \quad M_{ULT} = \phi f_y A_s \left(d - \frac{a}{2}\right)$$

$$= 0.9(60)(1.32)\left[\frac{7.1-0.5}{12}\right] = 39.2 \text{ KIP FT}$$

$$8\#4 \quad M_{ULT} = 0.9(60)(1.6)\left[\frac{5.6-0.5}{12}\right] = 36.7 \text{ KIP FT}$$

$$\overline{75.9 \text{ KIP FT} < \underset{\text{REQUIRED}}{89 \text{ KIP FT}}}$$

At the same time more east-west steel is provided just south of the elevator core than the design conditions require.

The finite element analysis made for the east half of the roof slab for the loading condition existing at failure (see, for example, Figures 1.3a and 1.3b in Part II, Section 1) are of some interest because the loads applied to the bay E5-E6-F6-F5 in those analyses are only slightly larger than the ultimate design loads considered here.

It appears that a finite element analysis for the ultimate design loads also would lead to design moments exceeding the capacity of the proportioned east-west steel which enters the elevator core.

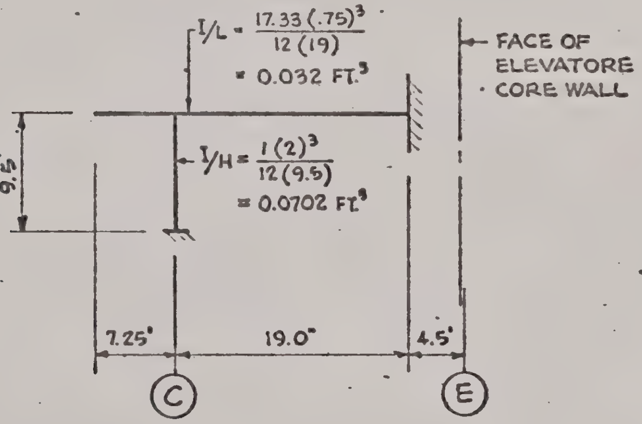
The preceding discussion has been based on the specified 60,000 psi minimum yield stress reinforcing steel. If one restricted steel stress to only 40,000 psi, the designed east-west steel entering the elevator core would not be sufficient for the ultimate design moments indicated either by the frame analysis made herein or by a finite element analysis.

Review indicates that all other east-west steel in the roof slab (both top and bottom) is properly proportioned for the stated design conditions.

Frame Analysis Along Line 6-1/2 on South Side of Elevator Core

Because of uncertainty regarding the influence of the brackets extending from the east and west elevator core walls, it would be necessary to make a finite element analysis to determine accurately, for the design conditions, the bending moments perpendicular to the south face of the elevator core wall. It was not convenient to make such an analysis in the investigation. Instead, an estimate was made of the minimum total ultimate load design moment which would have

to be resisted by the designed north-south top slab steel which enters the south wall of the elevator core. For this estimation it was assumed that the roof slab was fixed (by the two extending brackets) at the line of the south mechanical room wall beam. The total moment (for the entire 17 feet 4 inches width of the elevator core) was then calculated by a frame analysis.

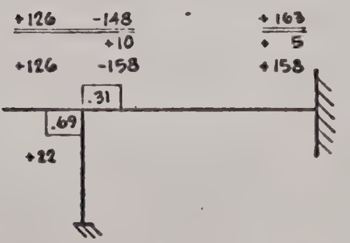

I/L = 17.33 (.75)³ / 12 (19) = 0.032 FT.³
FACE OF ELEVATORE CORE WALL
I/H = 1 (2)³ / 12 (9.5) = 0.0702 FT.³
9.5'
7.25'
19.0'
4.5'
C
E

$$\text{F.E.M.}_{\text{C-BRACKET}} = \frac{1.5(0.112)(17.33)(19)^2}{12} + \frac{1.8(0.100)(17.33)(19)^2}{12} + (0.75)^{*}$$

$$= 87.6 + 70.4 = 158.0 \ ^{\text{KIP FT}}$$

$$\text{CANT. MOM.} = \frac{1.5(0.094)(17.33)(7.25)^2}{2} + \frac{1.8(0.100)(17.33)(7.25)^2}{2} + (0.75)^{*}$$

$$= 64.2 + 61.4 = 125.6 \ ^{\text{KIP FT}}$$



As previously indicated the Code does not permit calculated elastic moments to be reduced for this particular slab.

The design provides 12 #4 and 6 #6 north-south top slab bars. The permissible ultimate bending moment capacity for these bars is:

$$
\begin{aligned}
M_{\text{ULT.}} &= \phi\, f_y A_s \left(d - \frac{a}{2}\right) \\
&= 0.9(60)\left[12(0.20) + 6(0.44)\right]\left[\frac{7.9 - 0.5}{12}\right] \\
&= 168^{\text{ KIP FT}}
\end{aligned}
$$

This calculation indicates that the total moment capacity is sufficient at this location. Based upon observation of the distribution of the total calculated moment for the failure condition (see Figure 1.3g in Part II, Section 1) it is questionable whether the 12 #4 north-south bars provided in the design would be sufficient to resist the ultimate load design moments as calculated by a more refined analysis.

With the possible exception of the 12 #4 bars noted above, review indicates that all north-south steel in the roof slab (both top and bottom) is properly proportioned for the stated design conditions.

Shear Stress Around Column E5

$$
\begin{aligned}
V_{\text{DESIGN}} &= 1.5(0.094)(400) + 1.8(0.030)(334)(0.75)^{*} \\
&\qquad + 1.8(0.100)(66)(0.75)^{*} \\
&= 56.4 + 13.5 + 8.9 \\
&= 78.8^{\text{ KIPS}}
\end{aligned}
$$

As noted in the concluding paragraph of page 4-3, the moments transferred into column E5 will depend upon whether or not the bracket and mechanical floor slab above column E5 is in place prior to the application of load on the roof slab.

The presence or absence of this bracket is even more significant in relation to the effective section which is available to resist shear forces induced at column E5. The effective shear section when brackets are not yet in place is significantly less than that which is available in the final condition. For the condition of no brackets, frame analyses (not presented here) indicate that the moments to be transferred into column E5 are:

$M_y = 23^{\text{kip ft.}}$ (due to unbalanced gravity load in analysis of frame along line E)

$M_z = 1^{\text{kip ft.}}$ (due to unbalanced gravity load in analysis of frame along line 5)

With these forces, and using the same shear stress calculation explained in Part II, Section 1A, the [illegible]mum ultimate load shear stress on a peripheral section out 3 inches from the column face is:

$$
\begin{aligned}
V_{max.} &= \frac{V}{A} + \frac{KM_yc}{J_y} + \frac{KM_zc'}{J_z} \\
&= \frac{78.8}{576} + \frac{0.4(23\times12)(9)}{35640} + \frac{0.4(1\times12)(15)}{76680} \\
& 0.137 + 0.028 + 0.001 \\
& 0.166 \text{ KSI}
\end{aligned}
$$

The ultimate shear stress permitted by the ACI 318-63 Code for the 3000 psi concrete specified for the roof slab is:

$$\begin{aligned} V_{\text{ULT. ALLOW}} &= 4\phi\sqrt{f'_c} \\ &= 4(0.85)(\sqrt{3000}) \\ &= 186 \text{ PSI} \end{aligned}$$

With the assumption that no load would act on the roof slab until the bracket was in place, the calculated maximum design load shear stress is well below that allowed by the Code.

Similarly, review of the slab conditions at other columns and at the elevator core walls indicates that maximum design load shear stresses are less than the value permitted by the Code.

B. THE SIXTEENTH FLOOR SLAB

Loadings

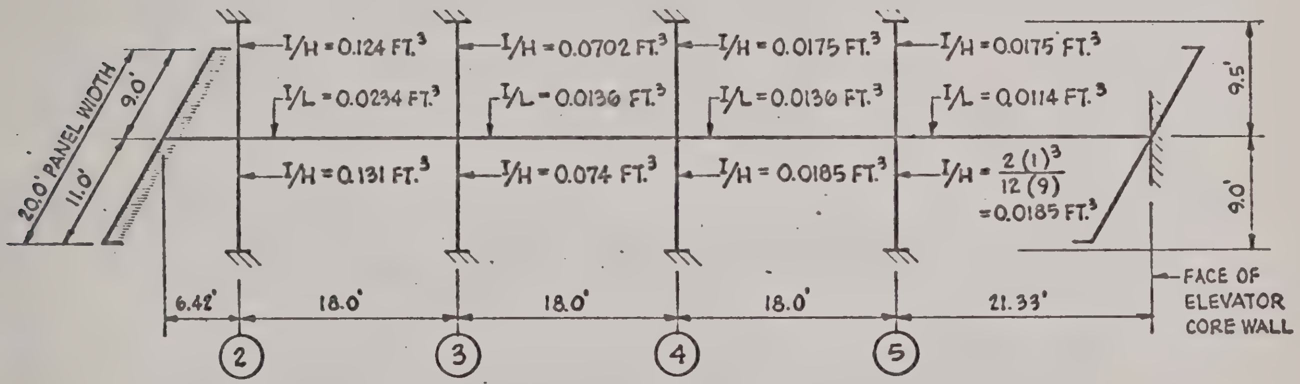
The design gravity loadings for the roof slab include the slab dead weight plus 20 pounds per square foot for partitions, etc. and 40 pounds per square foot live load. These loadings conform to the requirements in the Boston Building Code.

Thickness and Slab Reinforcement

The previous comments on page 4-3 regarding the design roof slab thickness around interior columns E4, E5, F4, F5 and E8, E9, F8, F9 are also applicable

to the 16th floor slab. Because the conditions are similar, analyses were made to determine whether the designed 16th floor slab reinforcement is sufficient for the stated design conditions. This review indicated that all slab reinforcement was properly proportioned. As indicated by the following analysis, however, the specified steel would not be sufficient given a steel yield stress of 40,000 instead of 60,000 psi.

Frame Analysis Along East-West Line E Columns



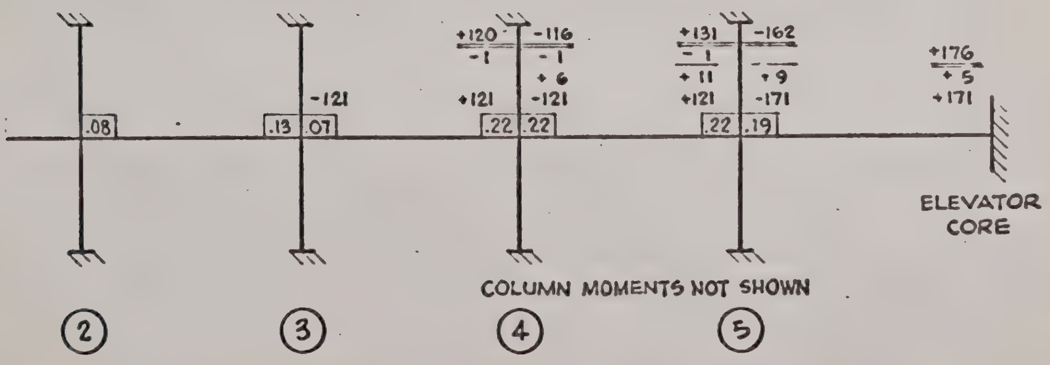
$$\text{F.E.M.}_{\text{5-ELEV.}} = \frac{1.5(0.094+0.020)(20)(21.33)^2}{12} + \frac{1.8(0.040)(20)(21.33)^2}{12} \times (0.75)^{*}$$

$$= 129.8 + 41.0 = 170.8^{\text{KIP FT.}}$$

$$\text{F.E.M.}_{5\text{-}4} = \frac{1.5(0.094+0.020)(20)(18)^2}{12} + \frac{1.8(0.040)(20)(18)^2}{12} \times (0.75)^{*}$$

$$= 92.3 + 29.2 = 121.5^{\text{KIP FT.}}$$

* LIVE LOAD REDUCTION FACTOR



At the "critical section" out 3-3/4 inches from the face of the wall the calculated bending moment is about 170 kip feet.

Since 11.83 feet of the total 20 feet panel width abuts the elevator core, the top steel bars entering the elevator core should have a moment resisting capacity of 11.83/20 x 170 = 101 kip feet. The permissible ultimate bending moment capacity for these bars is:

$$4\#6 \quad M_{ULT} = \phi f_y A_s \left(d - \frac{a}{2}\right)$$

$$= 0.9(60)(1.76)\left[\frac{7.1-0.5}{12}\right] = 52.3^{\text{KIP FT}}$$

$$11\#4 \quad M_{ULT} = 0.9(60)(2.2)\left[\frac{5.6-0.5}{12}\right] = 50.5^{\text{KIP FT}}$$

$$102.8^{\text{KIP FT}} > 101^{\text{KIP FT}}_{\text{REQUIRED}}$$

Thus the proportioned reinforcement is sufficient, given a design yield stress of 60,000 psi. It would not be sufficient given a stress of 40,000 psi.

Shear Stress Around Column E5

$$V_{DESIGN} = 1.5(0.094+0.020)(400) + 1.8(0.040)(400)(0.75)^{*}$$

$$= 68.5 + 21.6$$

$$= 90.1^{\text{KIPS}}$$

The moments induced into column E5 were calculated on the assumption that the 16th story column and roof slab would be in place prior to the application

of any load on the 16th floor slab. On this basis:

$M_y = 31^{\text{kip ft.}}$ (unbalanced gravity load moment induced into column E5, from previous analysis of frame along Line E)

$M_z = 28^{\text{kip ft.}}$ (unbalanced gravity load moment induced into column E5; from analysis of frame along Line 5, not presented here)

$$V_{max} = \frac{90.1}{576} + \frac{0.4(31 \times 12)(9)}{35640} + \frac{0.4(28 \times 12)(15)}{76680}$$

$$= 0.156 + 0.037 + 0.026$$

$$= 0.219 \text{ KSI} > 0.186 \text{ KSI ALLOWABLE}$$

This 18 percent overstress has not been listed as a design deficiency in Part I, Sections 3 and 4B because of three factors:

1. As illustrated by the analyses made for the failure conditions, the magnitude of M_y and M_z moments transferred into column E5 actually will be smaller than those derived from frame analyses such as used herein. While it is unlikely that the Structural Engineer made a finite element analysis during the design of the structure, the results of such an analysis probably would indicate transferred moments of only about one-half those used in the above shear stress calculation. If this be so, the calculated maximum shear stress would drop a value just about equal to that permitted by the Code.

2. Just as the ACI 318-63 Code does not prescribe the method of analysis to be used in the calculation of moment and shear forces, neither does it prescribe a procedure or formula for calculating the maximum shear stress. For the condition at issue here, the Code prescribes only that "the stresses on the critical section shall be investigated by a rational analysis, and the section proportioned" so that the maximum shear stress does not exceed 0.186 ksi. While it is believed that the formula used herein for the calculation of maximum shear stress is "rational," the Structural Engineer may feel that another procedure is equally rational and at the same time indicates slightly smaller maximum stresses.

3. In view of the two foregoing points, and considering that the Structural Engineer has not made his design calculations available to the Commission, a detailed investigation of the many possible options available to the designer under the Code would have to be made to show that this aspect of the design did (or did not) conform to the Code. Such an investigation was not within the scope of the Commission's studies.

The design shear condition at Column E5 is typical of that at column F5. It also is very nearly the same as those at columns E8, E9, F8, and F9. Accordingly, the same comments made regarding the slab section around column E5 apply as well to critical sections around these other columns. Review indicates that the design shear stresses at all other 16th floor slab locations are acceptable.

C. OTHER PORTIONS OF THE STRUCTURE

Typical Floor Slabs

The 16th down through the 3rd floor slabs all have the same design dimensions, design gravity loads, and proportioned reinforcing.

The design differences which must be considered in reviewing floor slabs down through the structure are:

1. The design dimensions of certain columns increase below the 11th floor slab. For example, the dimensions of columns E3, E4, E5, E8, E9, E10, F3, F4, F5, F8, F9, and F10 increase from 12 x 24 inches to 15 x 24 inches below the 11th floor, to 18 x 24 inches below the 8th floor, and to 21 x 24 inches below the 4th floor.

2. The specified concrete compressive strength at the 5th floor, and below, increases from 3000 to 4000 psi.

3. The application of design wind loads will cause bending moments to be induced into the columns and, consequently, into the floor slabs. When wind loads are considered the ACI 318-63 Code specifies that the design capacity shall be $U = 1.25\ (D+L+W)$, as opposed to $U = 1.5D+1.8L$ when wind is neglected. (The Boston Building Code prescribes a wind loading of 20 pounds per square foot of exposed elevation.)

An approximate wind load analysis indicates that the elevator core absorbs the major portion of any applied wind load, with the several floor slabs acting as diaphragms to distribute the loads to the core walls. Some small bending moments will be induced into the columns and floor slabs as a consequence of frame action under lateral loads, but the magnitude of these moments should not exceed the previously calculated unbalanced gravity load omoments. Accordingly, the previously analyzed gravity load condition would govern the design of each of the typical floor slabs.

The design differences which must be considered in reviewing floor slabs down through the structure are:

1. The design dimensions of certain columns increase below the 11th floor slab. For example, the dimensions of columns E3, E4, E5, E8, E9, E10, F3, F4, F5, F8, F9, and F10 increase from 12 x 24 inches to 15 x 24 inches below the 11th floor, to 18 x 24 inches below the 8th floor, and to 21 x 24 inches below the 4th floor.

2. The specified concrete compressive strength at the 5th floor, and below, increases from 3000 to 4000 psi.

3. The application of design wind loads will cause bending moments to be induced into the columns and, consequently, into the floor slabs. When wind loads are considered the ACI 318-63 Code specifies that the design capacity shall be $U = 1.25 (D+L+W)$, as opposed to $U = 1.5D+1.8L$ when wind is neglected. (The Boston Building Code prescribes a wind loading of 20 pounds per square foot of exposed elevation.)

An approximate wind load analysis indicates that the elevator core absorbs the major portion of any applied wind load, with the several floor slabs acting as diaphragms to distribute the loads to the core walls. Some small bending moments will be induced into the columns and floor slabs as a consequence of frame action under lateral loads, but the magnitude of these moments should not exceed the previously calculated unbalanced gravity load moments. Accordingly, the previously analyzed gravity load condition would govern the design of each of the typical floor slabs.

All comments made in regard to the design of the 16th floor slab are equally applicable to the 15th, 14th, 13th, and 12th floor slabs. Below the 12th floor the design shear capacity around the critical columns increases.

Columns

The design of column E5 from the roof slab to the footing was checked in detail and found to satisfy the Code provisions. Other columns, while reviewed in less detail, also appear to be properly designed.

Elevator Core Walls

The design of the elevator core walls was reviewed for resistance to gravity loads and gravity loads plus total wind load on the building. For both of these design conditions approximate analyses indicate that the proportioned core walls conform to Code requirements.

Foundations

Structural plan S-1 states that all footings shall be founded on naturally consolidated undisturbed soil capable of sustaining 10 tons per square foot or on sound, undisturbed rock (below elevation 137 feet) capable of sustaining 25 tons per square foot. The only footings specified to be below elevation 137 feet are those beneath the elevator core walls. These design assumptions were based upon results from a series of soil borings shown on the architectural plans.

A review of the design loads imposed upon the designed footings for the elevator core walls and for column E5 indicates conformity with the design assumptions. By appearance the specified footings for other columns also would seem to be satisfactory.

D. DETAILING OF REINFORCEMENT

Several inconsistencies and/or deficiencies were noted regarding the detailing of the reinforcement by the Structural Engineer. The most significant of these are noted in Part I, Section 4B and a few additional ones are mentioned in Part II, Section 3A.

5.

THE ROOF AND MECHANICAL FLOOR SLABS

A. MAIN ROOF SLAB

Design Dimensions

The main roof slab design dimensions are extracted from the structural and architectural plans in Figure 5.1. For reference purposes the columns below, elevator core walls, and mechanical floor above are also indicated.

Placement and Collapse

The roof slab was placed in two parts. The Supplier's delivery records and invoices show 218 cubic yards of concrete delivered on December 3, 1970, and 95 cubic yards delivered on December 9, 1970, for a total of 313 cubic yards. According to testimony placement started at the west edge and proceeded east.

On the basis of the design plans, calculated volumes in the roof slab are:

From	To	Volume (cubic yards)
West edge	Line 7	134
West edge	Line 6	181
West edge	Line 5	217
West edge	Const. jt. as per Mr. Kanieff's testimony	245
West edge	Line 4	248
West edge	Line 3½	262
West edge	East edge	316

These volumes account for the stairwell openings and the elevator core but not for mechanical openings.

The volume calculations indicate that Mr. Kanieff, and other employees of the Structural Subcontractor, were mistaken in their testimony that the roof slab construction joint was at line 3½ on the exterior and line 4½ on the interior. Rather, the calculations show that the construction joint must have been in the vicinity of line 5. Three possibilities for the actual location are discussed in Part 1, Section 3A. The position which is believed to be the most likely possibility is shown in Figure 5.2.

Figure 5.2 also shows the extent of the roof slab collapse and the location of four collapsed roof slab pieces which were saved. The photograph in Figure 5.3 shows the extent of the collapse in the top stories as seen on January 27, 1971. In the days following the collapse the drooping slab pieces seen in Figure 5.3 were cut loose. The roof slab piece on the south side of the elevator core was cut loose on February 3, 1971. Part of this piece was saved and marked π15. Piece π15 and the other three saved pieces, π9, π11, and π20, are carefully detailed and photographed in Appendix II.1. Another saved piece,

Π4, has a top surface which is characteristic of the roof slab but it could not be positively identified as coming from the roof.

Concrete and Reinforcement

Tables 1 and 2 in Part I summarize the test results obtained from cores taken from the roof slab. The evaluation of these results is summarized in Part I, Section 3A and presented in more detail in Part II, Section 2.

A comparison of the reinforcemement found in pieces Π9, Π11, Π15, and Π20 with that specified on the structural plans and bar placing drawings is contained in Part II, Section 3D.

After the collapse, the Owners were requested to authorize certain notching of the uncollapsed portion of the roof slab to determine the quantity and position of reinforcing steel around the southwest corner of the elevator core and around column E8. This investigation, which would have been conducted after completion of the planned shoring program, was not authorized.

Openings

Figures 5.4, 5.5, and 5.6 concern openings in the roof slab. Figure 5.4 shows openings as indicated on the structural plans. The reinforcing steel was detailed, the bar placing drawings were prepared, and the steel was supplied on the basis of these openings. Figure 5.5 shows openings indicated on the revised mechanical plans (prepared by Mr. Joseph Schneider for the Mechanical Subcontractor). Figure 5.6 shows actual openings, as measured, in the uncollapsed portion of the slab. The basis for the actual openings has not been determined. The photograph in Figure 5.7 shows actual openings along line 9 between columns E9 and F9.

Slab Thickness

Measured roof slab thicknesses are shown in Figure 5.6 for the uncollapsed portion and for the collapsed pieces π9, π11, π15, and π20. In all cases measured thickness equalled or exceeded that specified on the plans.

The architectural plans call for the roof slab to slope two inches to indicated drains. On the uncollapsed portion of the roof two drains are present, one near column E10 and one near column F10. In both number and position these actual drains do not conform to the architectural plans.

Since the Structural Subcontractor testified that the bottom surface of the roof slab was constructed level and that slopes to drain were obtained by increasing slab thickness toward the edges of the slab, it would seem that a definite pattern of changing slab thickness should exist. Measured slab thickness at openings and at locations where concrete cores were taken did not indicate such a pattern. Thicknesses near the outside edges of the slab were not greater, and in some cases less, than those in interior bays. Since no thickness deficiency was found, a detailed survey of the uncollapsed roof slab was not made.

Loadings

In addition to self weight the roof slab was subjected to superimposed gravity loads consisting of the weight of the freshly placed mechanical room floor slab, walls and brackets; the formwork for these placements (see Part II, Figures 7.16, and 5.3); the construction materials and equipment resting on the roof slab; and possibly a very small quantity of snow.

Figure 5.8 shows the construction materials and equipment resting on the southwest (uncollapsed) side of the roof. Essentially no materials or

equipment were stored on the northwest side of the roof. These observations of conditions on the west side are consistent with the fact that the materials hoist and crane were located on the south side of the building.

Testimony has indicated that two boilers were temporarily stored on the south side of the roof somewhere between lines 5 and 7. They were eventually to have been installed on the east side of the mechanical room floor slab. Mr. Paolini testified that after collapse of the roof slab onto the 16th floor one boiler was teetering on the edge of the break line and the other was down on the 16th floor. Both boilers fell in the general collapse. For purposes of analysis their positions were assumed to be as shown in Part II, Figure 1.1.

The boilers were made by the Burnham Corporation, Irvington, New York. Base dimensions of one of the boilers which was removed from the debris were 53 x 101 inches. A name plate on the boiler identified it as a Burnham Golden Cube, 3 path forced draft generator, gross rating 100 HP, serial numbers PB4642NS and NB7601. Gross shipping weight was 7950 pounds each.

The compressor which fell from the roof during the collapse of the roof slab onto the 16th floor slab was a Schramm Air Compressor with a single axel and a trailer hitch. It was leased to the job by Crane Service Corporation, Boston, who gave the unit weight as 2475 pounds. Testimony indicated that the compressor fell from the south side of the roof somewhere between lines 4 and 6. In the structural analyses it was assumed to be located as shown in Part II, Figure 1.1.

There is no reason to suspect that the storage of miscellaneous construction materials on the southeast side of the roof slab was any different from that shown for the southwest side in Figure 5.8. However, because this evidence is only circumstantial, only 1000 pounds of materials are assumed to have been

present on the southeast side for purposes of structural analysis. This assumed loading is positioned as shown in Part II, Figure 1.1.

In the structural analyses no snow accumulation was assumed to be present on the roof slab. If any snow was present it must have been very little, as Weather Bureau records for the day of the collapse show an accumulated depth of snow at Logan Airport of only 1 inch. The photo in Figure 5.8a shows that no snow was present on the roof on February 3, 1971, although some depth of snow is evident on the ground in areas surrounding the site. No snow is evident in the photograph of Figure 5.3, which was taken on January 27, 1971.

In the structural analysis it was determined that the combined influence of all stored construction materials and equipment on the maximum shear conditions around column E5 was less that 2% of the influence due to self weight and the mechanical room construction loads.

B. MECHANICAL ROOM (PENTHOUSE) FLOOR SLAB

Design Dimensions

The mechanical room floor slab design dimensions, as extracted from the structural plans, are shown in Figure 5.1. Both the width and length dimensions of the floor slab (and supporting walls) differ on the structural and architectural plans, the width being 4 inches greater and the length being 20 inches smaller on the architectural plans. Measurement of the uncollapsed portion conforms with the structural plans.

The finished elevation of the mechanical room floor slab was designed to be 5 feet 2 inches above the roof slab. The brackets which extend out from columns E4, E5, E8, E9, F4, F4, F8, and F9 as well as the brackets which extend out

from the elevator core walls are designed to be monolithic with the mechanical floor slab and the roof slab. The 8 inch thick side and end wall beams are designed to be monolithic with the mechanical floor but not with the roof slab. A detail on the structural plan calls for a $\frac{1}{2}$ inch thick filler at the top of the roof slab under the wall beams.

Placement and Collapse

The mechanical room floor slab was placed on January 25, 1971. Testimony indicates that placement began at the west edge at about 10:00 a.m. and proceeded east, casting the slab, wall beams, and brackets together in a continuous fashion until a coffee break at about 3:00 p.m.

Unlike the wall beams which were to support the mechanical floor at its perimeter, the 8 inch thick walls which surround the stairwell are designed to be monolithic with the roof slab. The design calls for a $\frac{1}{2}$ inch filler vertically between the end wall beam of the mechanical room and the walls which surround the stairwell. It has not been established whether the walls surrounding the stairwell were cast prior to January 25, but at the finished level of the mechanical room floor there is no evidence of a $\frac{1}{2}$ inch filler between the walls and end wall beam.

On the basis of the structural plans, the calculated volumes in the walls, wall beams, brackets, and mechanical floor slab are:

Item	Volume (cubic yards)
8 inch thick walls surrounding west stairwell	6
Mechanical floor slab, wall beams, brackets from west edge to 3 feet east of line 5	106
Elevator core walls from roof to mechanical floor	10

These volumes do not account for mechanical openings in the floor slab or for openings in the 8 inch thick walls and wall beams.

The Supplier's delivery records indicate that 140 cubic yards of concrete were delivered on January 25, 1971. Subsequent to the Supplier's testifying before the Commission, a telephone inquiry revealed that part of the 140 cubic yards was not placed. According to the Supplier the last load, truck #15, with 10 cubic yards returned to the plant. Similarly truck #14 with 10 cubic yards and truck #13 with 9 cubic yards also returned to the plant full. Truck #12 was in the process of being unloaded when the failure occurred. This truck returned to the plant with about 1 cubic yard left in the truck. Accordingly, about 110 cubic yards were unloaded at the site. Assuming that there was 1 cubic yard in the crane bucket which had to be emptied prior to rescue of the three marooned workmen, it appears that about 109 cubic yards of concrete were placed prior to the collapse.

In comparing the volume placed with the calculated volumes, two additional items must be noted. First either the top story column E5 or F5 with the bracket above was saved from the collapse. Concrete had been placed in this bracket, clearly indicating that placement had reached line 5. Second, Messrs.

Niro and Oliva, who were employed as cement finishers, testified that they had just finished sweeping the second set of drains east of the elevator core prior to the coffee break at about 3:00 p.m. They stated that the slab was finished a few feet beyond the second set of drains on a line extending directly north and south. Subsequent to their testimony, Mr. Louis J. Moccaldi testified privately regarding the location of drains in the mechanical floor slab. Mr. Moccaldi, who personally placed these drains, indicated that they were as shown in Figure 5.9. Both the number and position of these drains do not conform to the revised mechanical plans (prepared by Mr. Joseph Schneider for the Mechanical Subcontractor) or to the records in the Building Department which formed the basis for approval of drains.

For the majority of the structural analyses of the east half of the roof slab it was assumed that the mechanical floor slab placement had extended 3 feet beyond line 5 as shown in Figure 5.9. As indicated by one analysis, where placement was assumed to extend only to line 5, the maximum shear conditions around column E5 are not drastically sensitive to the precise limit of placement.

Figures 5.3 and 5.10 show the extent of the collapse at the mechanical room floor level.

Openings

Figure 5.10 shows the actual openings, as measured in the field, in the uncollapsed portion of the mechanical roof floor slab. There are no floor drains in this portion of the slab.

Openings for the mechanical floor slab as indicated on the structural and revised mechanical plans differ significantly from those shown in Figure 5.10.

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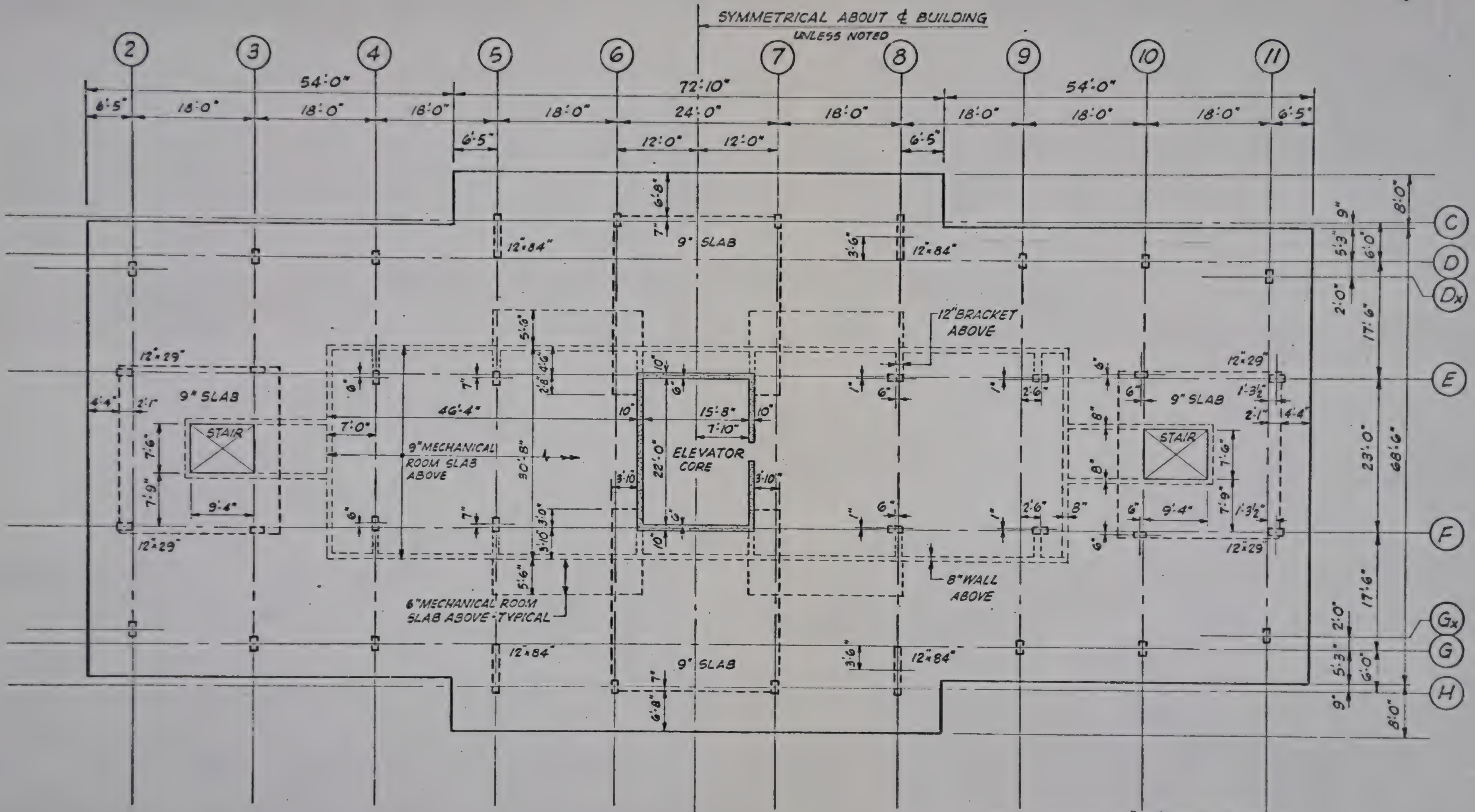


ALL COLUMNS 12"×24" UNLESS NOTED
TYPICAL SLAB THICKNESS = 7½"

FIGURE 5.1 SCALE: 1/16" = 1'-0"
ROOF SLAB DESIGN DIMENSIONS

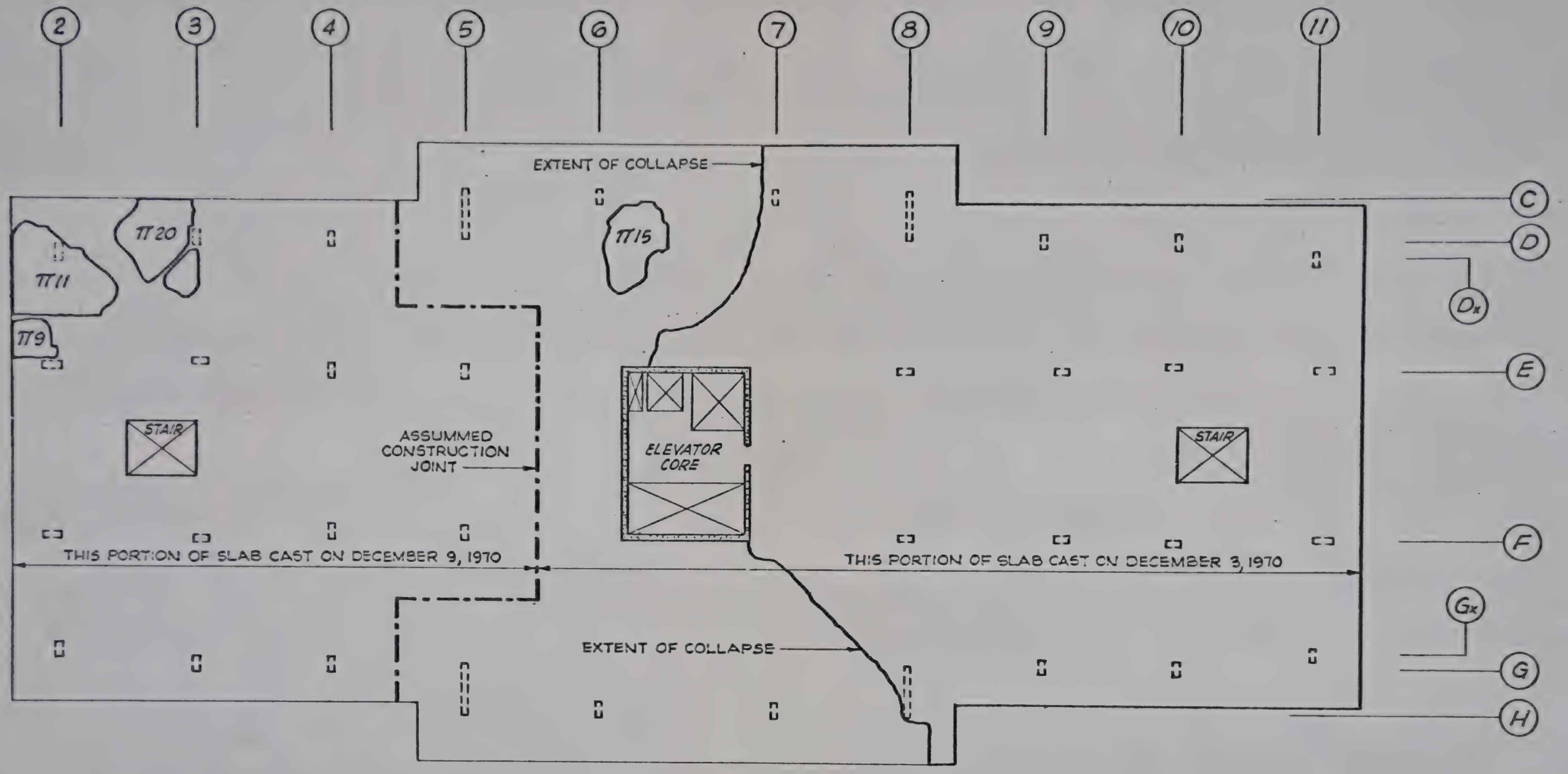
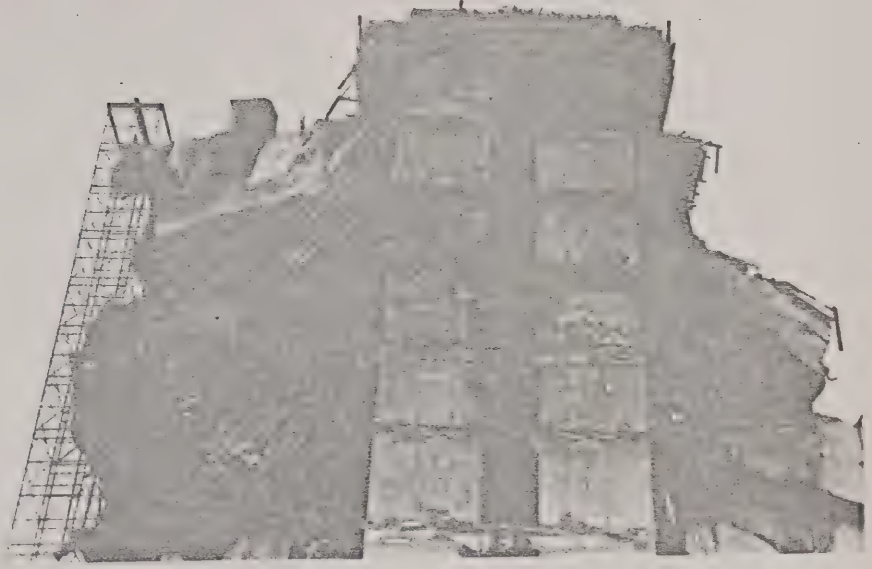

2
3
4
5
6
7
8
9
10
11
EXTENT OF COLLAPSE
C
D
Dx
π20
π11
π15
π9
E
STAIR
ASSUMED CONSTRUCTION JOINT
ELEVATOR CORE
STAIR
F
THIS PORTION OF SLAB CAST ON DECEMBER 9, 1970
THIS PORTION OF SLAB CAST ON DECEMBER 3, 1970
Gx
EXTENT OF COLLAPSE
G
H

FIGURE 5.2 SCALE : 1/16" = 1'-0"

ROOF SLAB GENERAL INFORMATION ON CASTING, COLLAPSE AND RECOVERED PIECES

FIGURE 5.3 EXTENT OF COLLAPSE OF TOP STORIES



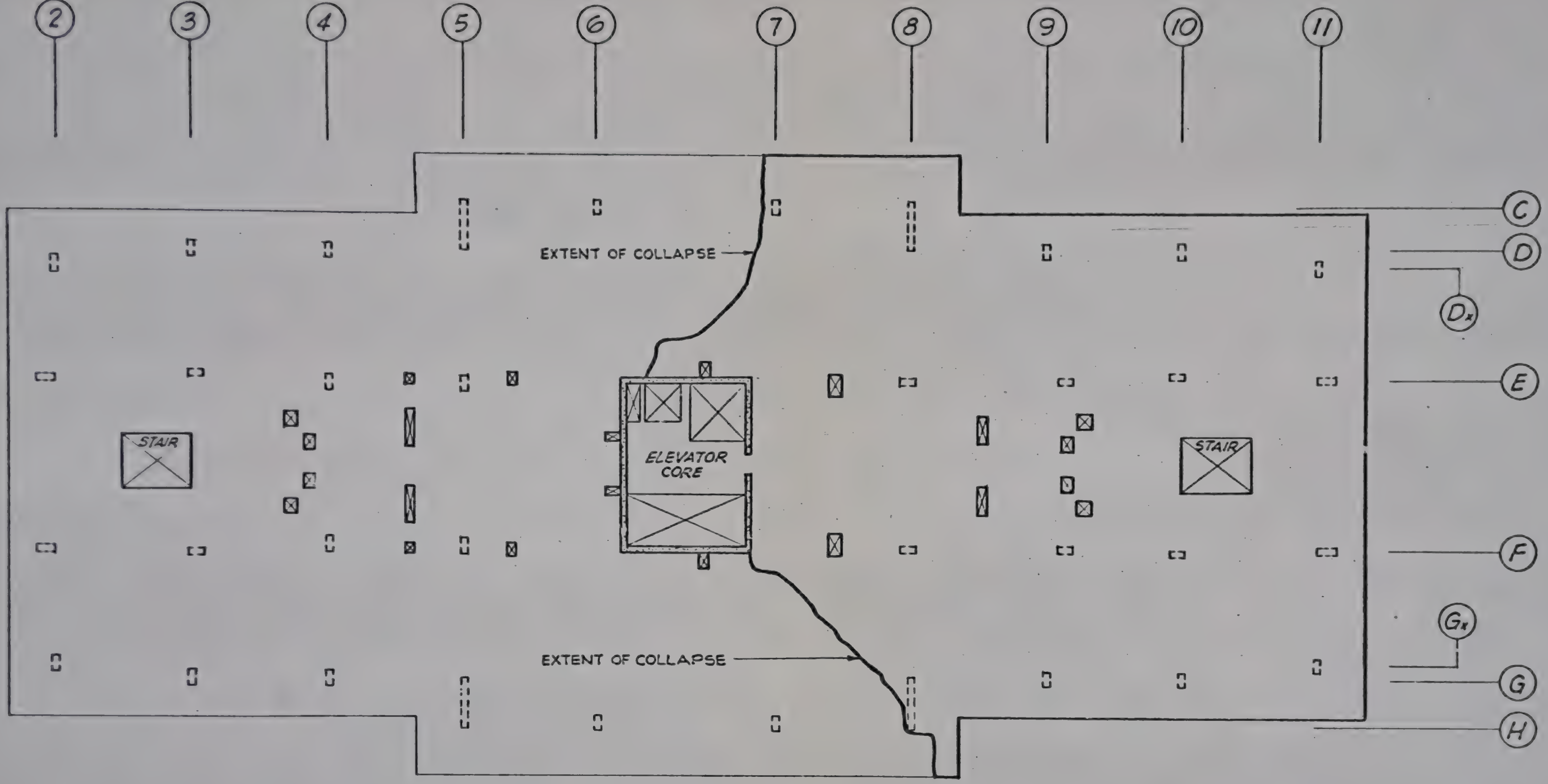


FIGURE 5.4 SCALE : 1/16" = 1'-0"
ROOF SLAB OPENINGS SHOWN
ON STRUCTURAL PLANS

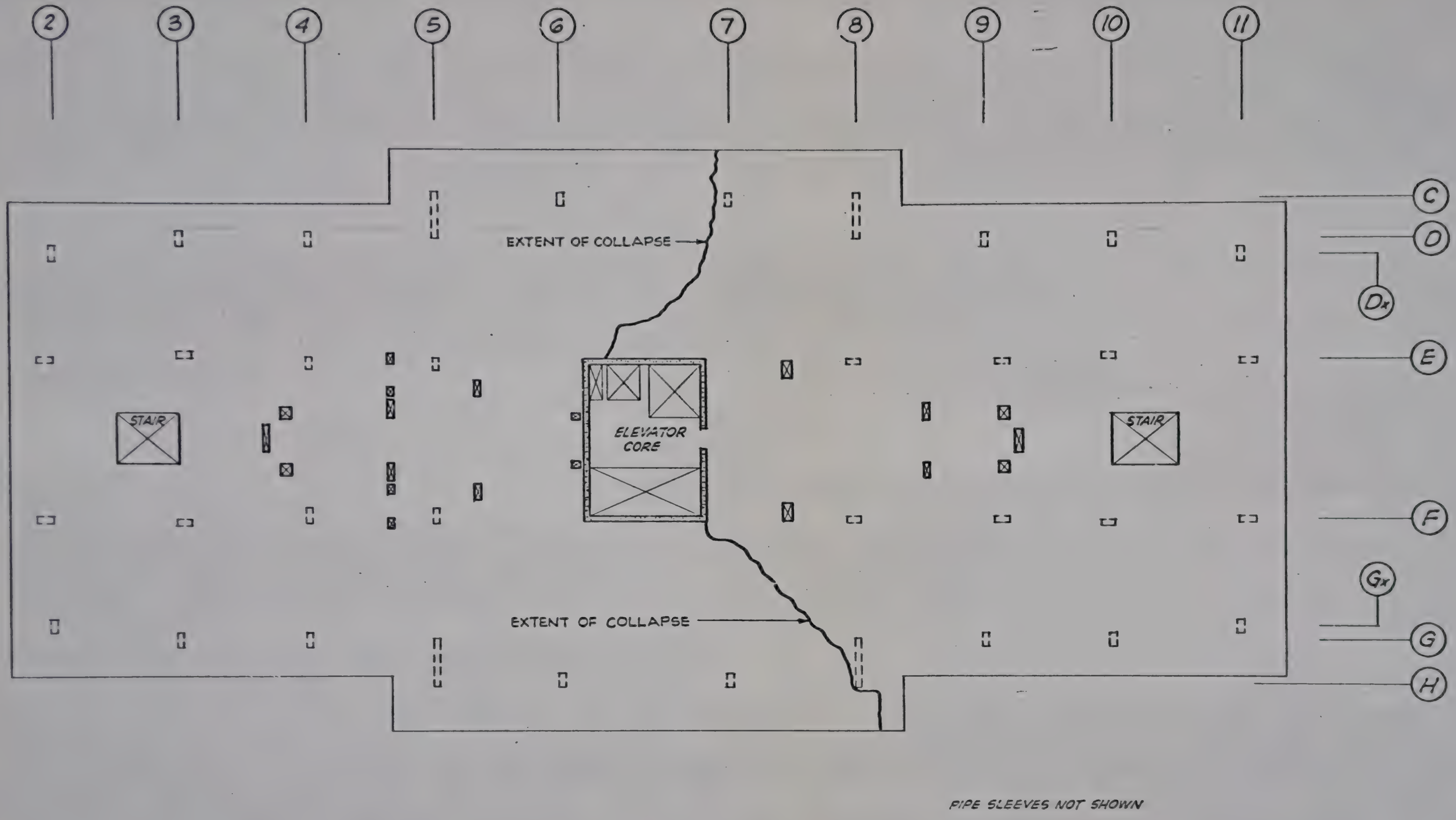
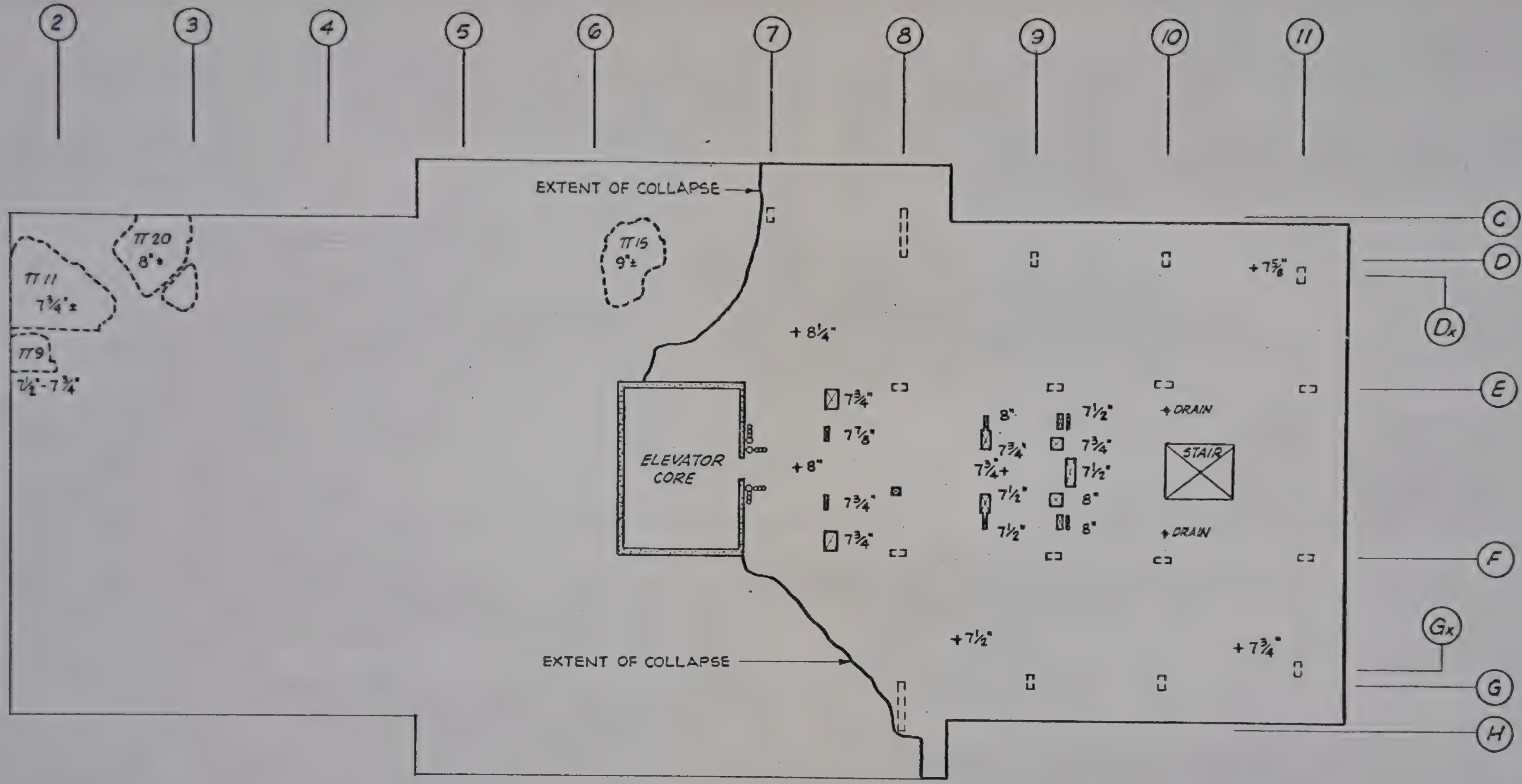


FIGURE 5.5 SCALE: 1/16" = 1'-0"
ROOF SLAB OPENINGS SHOWN ON MECHANICAL PLANS



ALL PIPE SLEEVES NOT SHOWN
+ CORE LOCATIONS

FIGURE 5.6 SCALE: 1/16" = 1'-0"
ROOF SLAB OPENINGS AND THICKNESSES IN THE UNCOLLAPSED STRUCTURE

FIGURE 5.7 OPENINGS IN ROOF SLAB BETWEEN COLUMNS E9 and F9

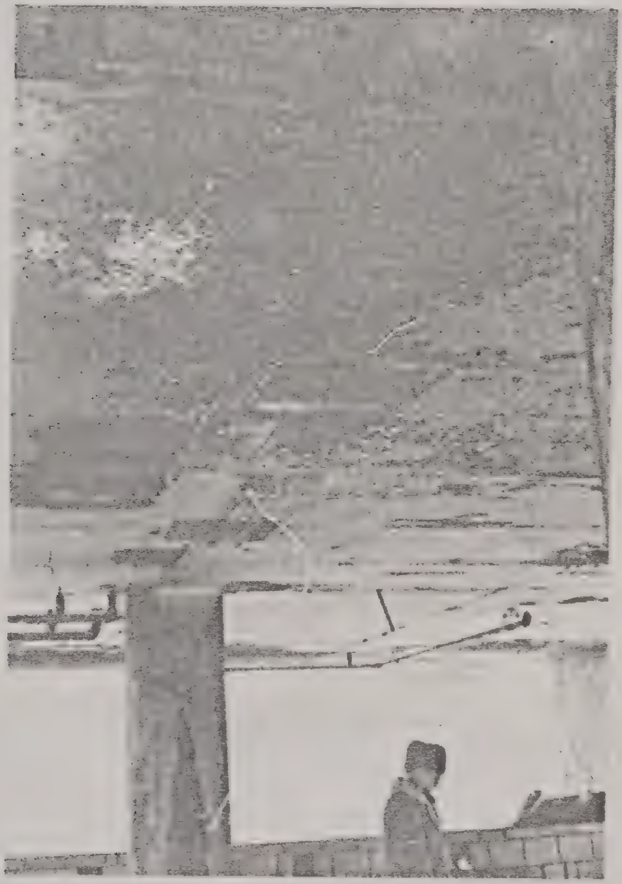
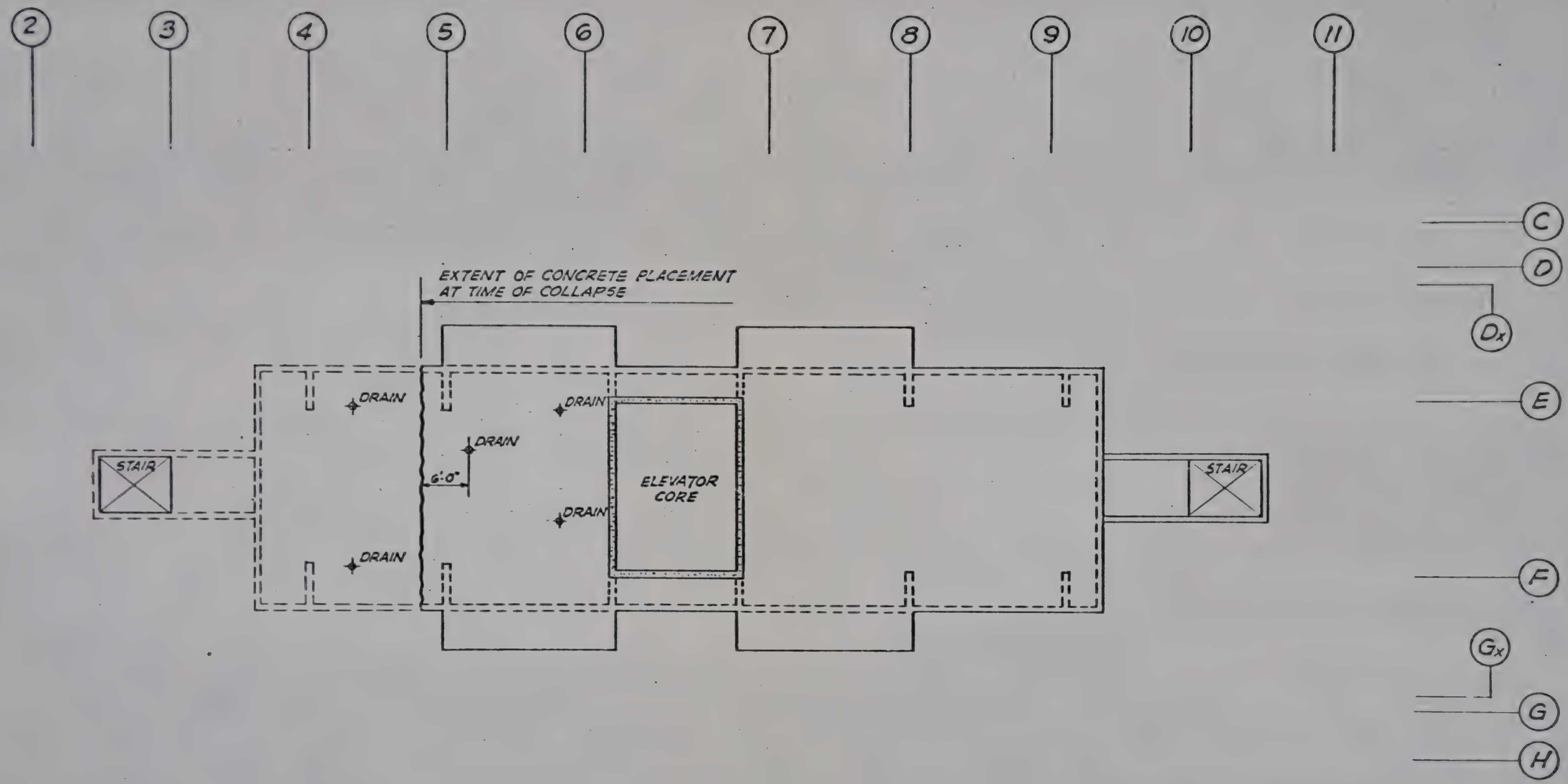


FIGURE 5.8 MATERIALS AND EQUIPMENT ON SOUTHWEST PORTION OF THE ROOF SLAB

(a) Photograph Taken February 3, 1971

(b) Photograph Taken March 1, 1971



MR. LOUIS MOCCALDI TESTIFIED THAT ONLY FIVE DRAINS WERE PLACED IN THE MECHANICAL ROOM FLOOR SLAB AND THAT HE PLACED THEM. THE LOCATIONS SHOWN ARE THOSE INDICATED BY MR. MOCCALDI. NO DRAINS WERE PLACED IN THE UNCOLLAPSED PORTION OF THE SLAB.

FIGURE 5.9 SCALE: 1/16" = 1'-0"
MECHANICAL FLOOR SLAB
FLOOR DRAIN LOCATIONS

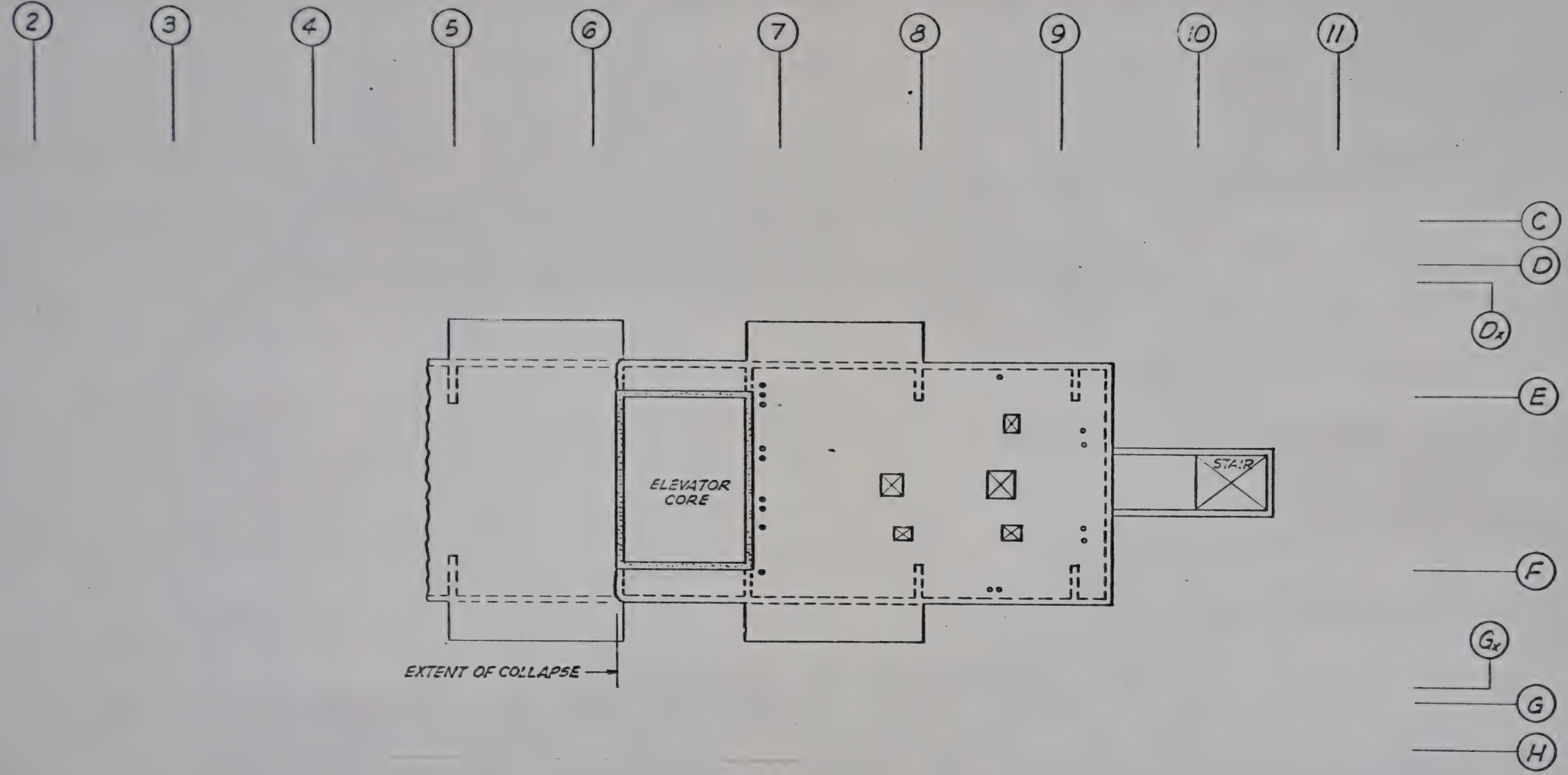


FIGURE 5.10 SCALE : 1/16" = 1'-0"

MECHANICAL FLOOR SLAB OPENINGS IN THE UNCOLLAPSED STRUCTURE

6.

THE 16TH FLOOR SLAB

Design Dimensions

The 16th floor slab design dimensions are extracted from the structural and architectural plans in Figure 6.1. These plan dimensions are also typical of the 3rd through 15th floor slabs except that certain column dimensions become slightly larger in lower stories and the location of these column centers changes slightly.

Placement and Collapse

According to the testimony, placement of concrete in the 16th floor slab started at the west edge and proceeded east. The Testing Laboratory's site inspection reports indicate that the slab was placed on two different days, November 18 and November 20, 1970. For these days the Concrete Supplier's delivery records and invoices show delivery of 212 cubic yards and 76 cubic yards of concrete, respectively. The Testing Laboratory's site inspection report for November 20, 1970, states that the 76 cubic yards were used in the 16th floor slab, in stairs, and in parapets. Calculation indicates that less than 67 cubic yards would have been required to complete the 16th floor slab.

On the basis of the design plans, volumes of the 16th floor were calculated.

(The Structural Subcontractor's shop drawing for dimensioning of the typical floors conformed to the design plans.) The calculated volumes are:

From	To	Volume (cubic yards)
West edge	Line 7	119
West edge	Line 6	160
West edge	Line 5	193
West edge	Line 4	221
West edge	Line 3	247
West edge	East edge	279

These volumes account for the stairwell openings and the openings in the elevator core but not for mechanical openings.

By comparing the calculated volumes with the concrete deliveries, it appears that a construction joint must have been placed in the vicinity of line 4½. An assumed joint location is shown in Figure 6.2.

Figure 6.2 also shows the extent of the 16th floor slab collapse and the location of one collapsed piece, marked Π71, which is believed to have come from the 16th floor. Piece Π71 is carefully detailed in Appendix II.1.

Concrete Test Results

Tables 2.5 and 2.6 summarize the test results obtained from cores taken from the 16th floor slab. Table 2.5 which includes results obtained from the uncollapsed portion of the slab, pertains to the November 18, 1970, placement. This placement would have included the concrete around columns E5 and F5. Table 2.6 includes results obtained for piece Π71 which would have been cast

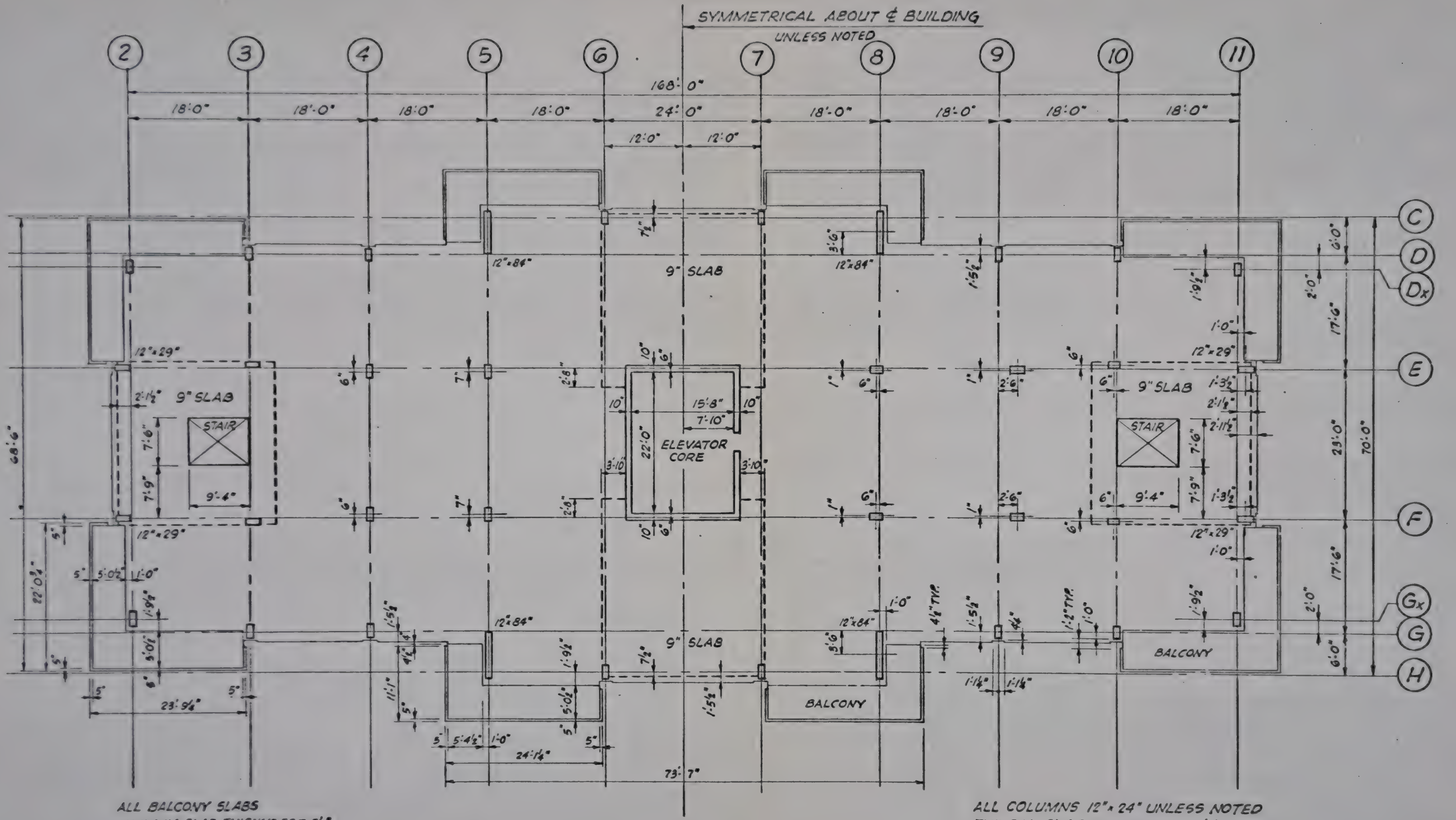
on November 20, 1970. These tests are described in more detail in Part II, Section 2 and the laboratory test reports are contained in Appendix II.2.

Openings

Figures 6.3, 6.4, an 6.5 concern openings in the 16th floor slab. Figure 6.3 shows openings as indicated on the structural plans. The reinforcing steel was detailed, the bar placing drawings were prepared, and the steel was supplied on the basis of these openings. Figure 6.4 shows openings indicated on the revised mechanical plans (prepared by Mr. Joseph Schneider for the Mechanical Subcontractor). Figure 6.5 shows actual openings, as measured, in the uncollapsed portion of the slab. The basis for the actual openings has not been determined.

Slab Thickness

Measured 16th floor slab thicknesses are shown in Figure 6.5 for the uncollapsed portion and for collapsed piece Π71. In some places actual slab thickness is less than that specified on the structural plans.



ALL BALCONY SLABS
MAXIMUM SLAB THICKNESS = 5½"

ALL COLUMNS 12" × 24" UNLESS NOTED
TYPICAL SLAB THICKNESS = 7½"

FIGURE 6.1 SCALE: 1/16" = 1'-0"
16TH FLOOR SLAB DESIGN DIMENSIONS

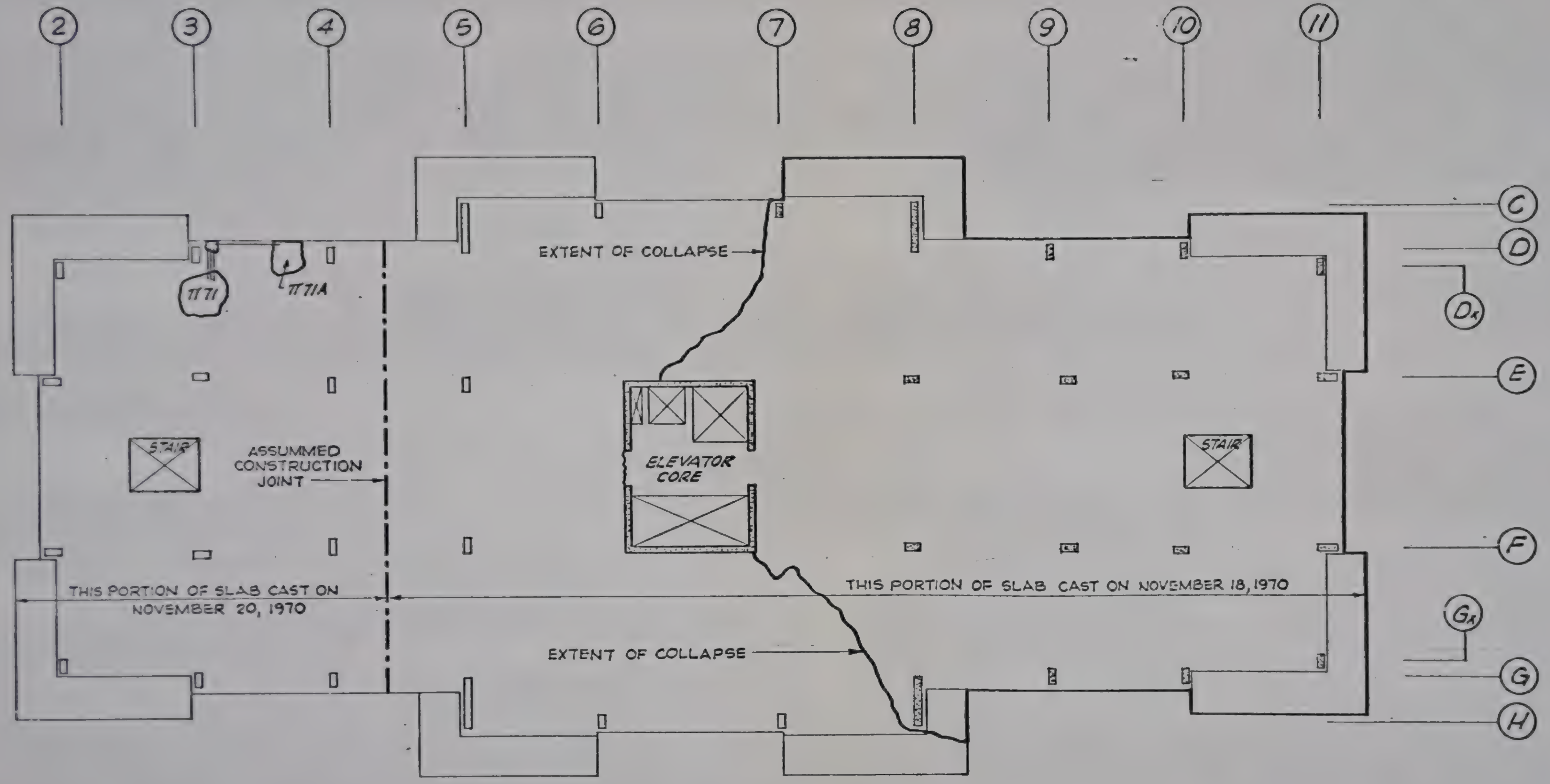

2
3
4
5
6
7
8
9
10
11
C
D
D_x
E
F
G_x
G
H
π71
π71A
EXTENT OF COLLAPSE
STAIR
ASSUMMED CONSTRUCTION JOINT
ELEVATOR CORE
STAIR
THIS PORTION OF SLAB CAST ON NOVEMBER 20, 1970
THIS PORTION OF SLAB CAST ON NOVEMBER 18, 1970
EXTENT OF COLLAPSE

FIGURE 6.2 SCALE : 1/16" = 1'-0"
16TH FLOOR SLAB GENERAL INFORMATION ON CASTING, COLLAPSE AND RECOVED PIECES

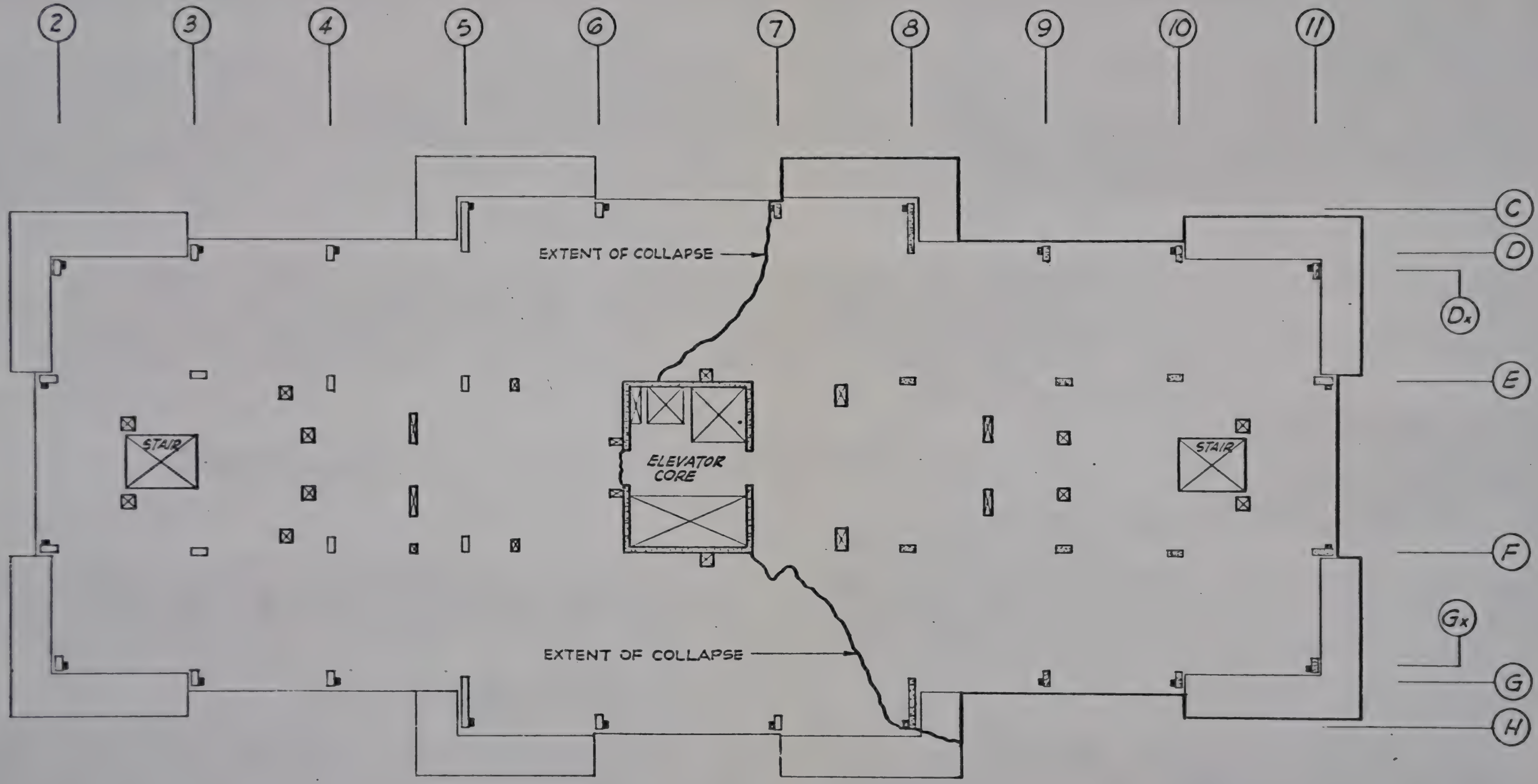


FIGURE : 6.3 SCALE : 1/16" = 1'-0"
16TH FLOOR SLAB OPENINGS SHOWN ON STRUCTURAL PLANS

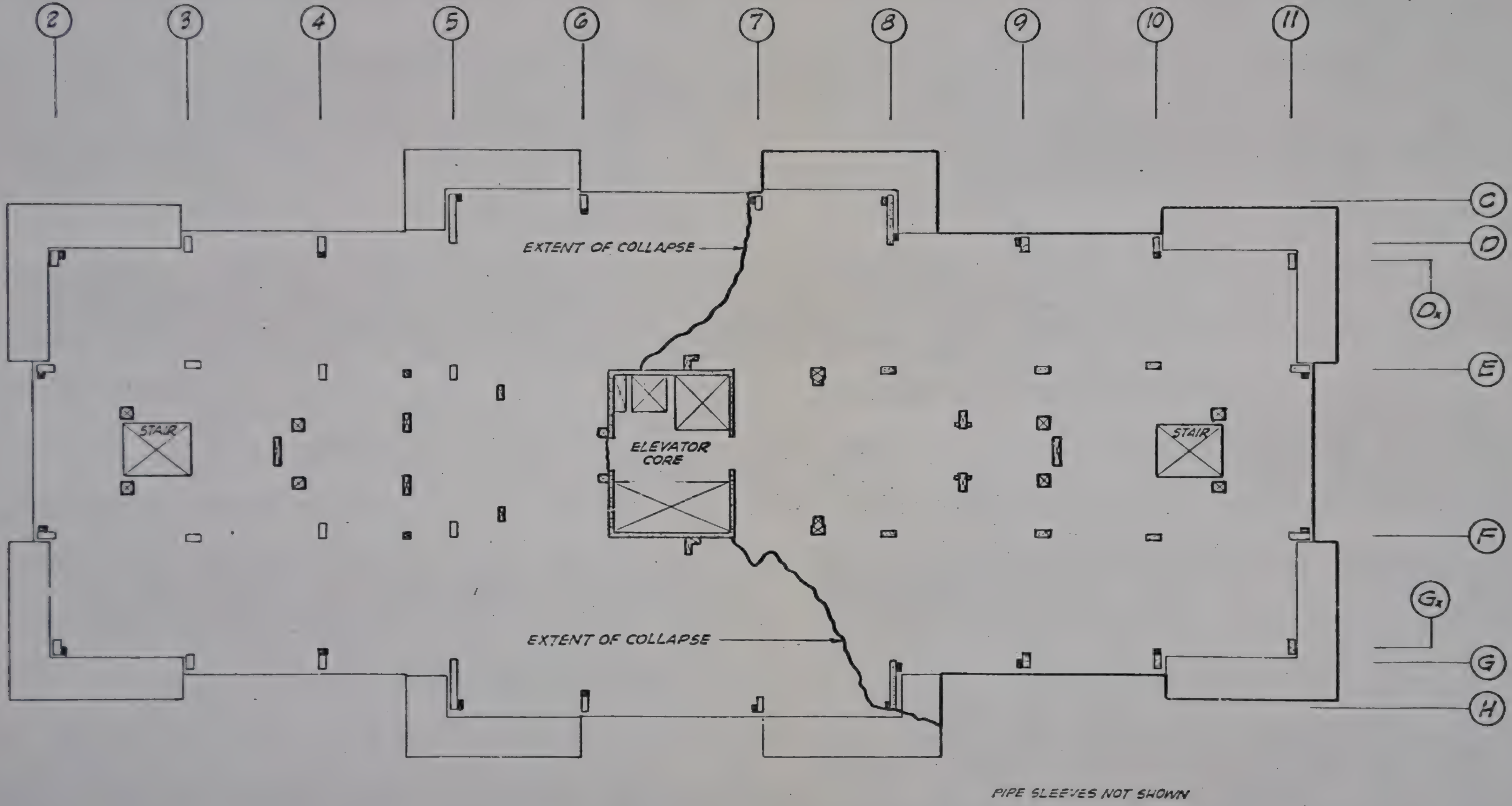
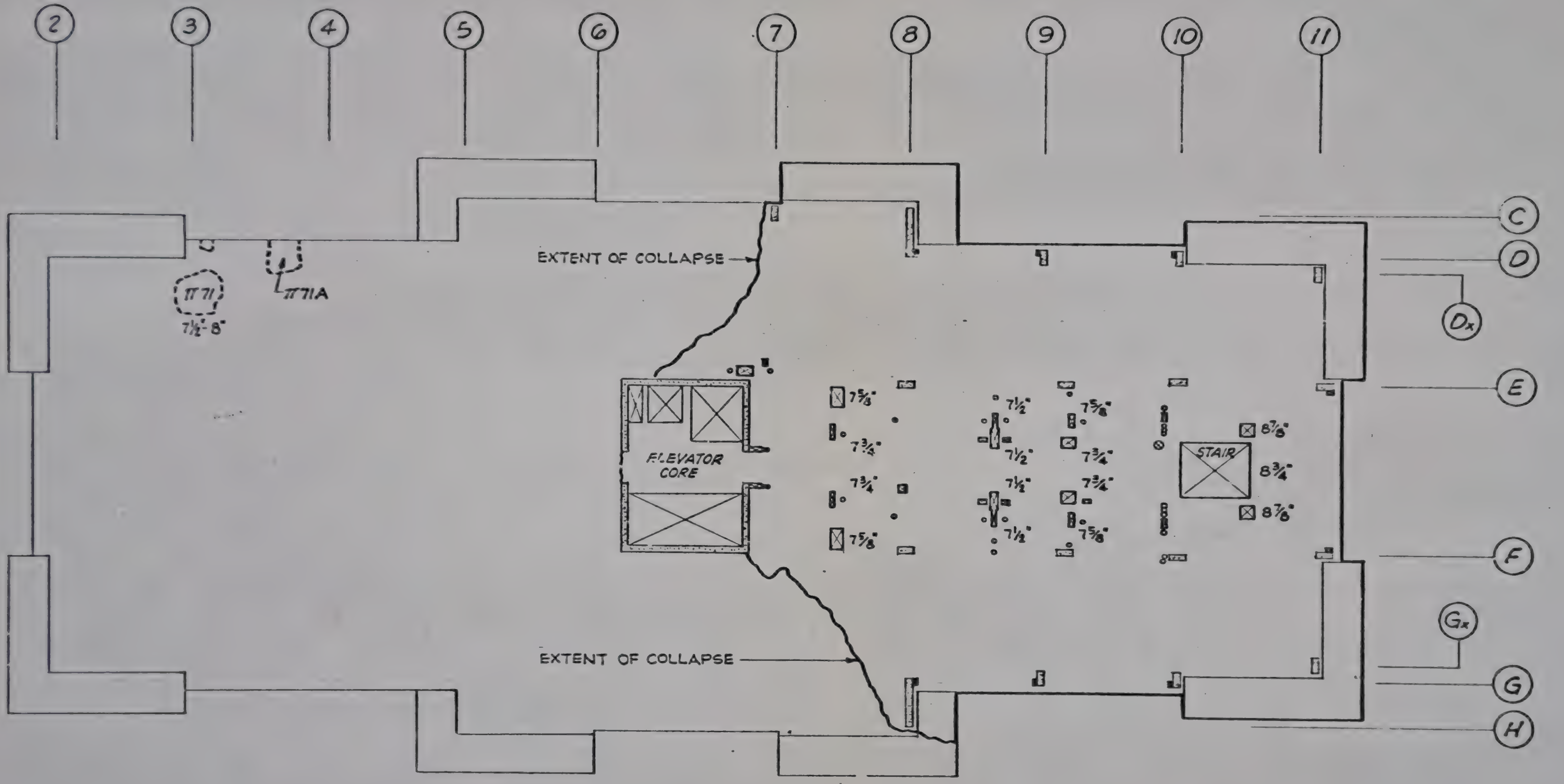


FIGURE 6.4 SCALE: 1/16" = 1'-0"
16TH FLOOR SLAB OPENINGS SHOWN ON MECHANICAL PLANS



ALL PIPE SLEEVES NOT SHOWN

FIGURE 6.5 SCALE: 1/16" = 1'-0"
16TH FLOOR SLAB OPENINGS AND THICKNESSES IN THE UNCOLLAPSED STRUCTURE

7.

SHORING

A. 16TH FLOOR TO ROOF

Structural plans S-1 and S-8 note that temporary construction loading on the roof slab should not exceed 30 pounds per square foot, except on the south side of the roof between lines 5 and 8. Additionally, plan S-8 specifically called for the placement of temporary shores under the roof slab below the cantilever brackets which extend outward along columns 4, 5, 8, and 9 from columns E4, E5, E8, E9, F4, F5, F8, and F9. These shores were to assist in the distribution of the construction loads induced during and after placement of the mechanical room walls and floor slab.

The shoring provided under the uncollapsed western half of the roof was examined in the days immediately following the collapse. This examination revealed placement of the nineteen 4 x 4 inch wood shores shown in Figure 7.1. Figures 7.2 through 7.14 are photographs taken on February 6 and 8, 1971, which show the construction conditions which prevailed on the west side of the 16th story.

In Part I, Section 3A the testimony of eyewitnesses regarding the shoring present under the collapsed east side of the roof slab is summarized. This testimony is contained in Appendix II.4. These witnesses agreed that there was a series of shores around the perimeter of the east side, some of which are visible in

7.

SHORING

A. 16TH FLOOR TO ROOF

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The shoring provided under the uncollapsed western half of the roof was examined in the days immediately following the collapse. This examination revealed placement of the nineteen 4 x 4 inch wood shores shown in Figure 7.1. Figures 7.2 through 7.14 are photographs taken on February 6 and 8, 1971, which show the construction conditions which prevailed on the west side of the 16th story.

In Part I, Section 3A the testimony of eyewitnesses regarding the shoring present under the collapsed east side of the roof slab is summarized. This testimony is contained in Appendix II.5. These witnesses agreed that there was a series of shores around the perimeter of the east side, some of which are visible in

Figures 2 and 6 of Part I. These perimeter shores were used to support a weather protection screen which terminated at the line of shores in bay 7-8 on the west side. From the shores and tarpaulins visible in Figures 7.8 to 7.10 it seems likely that the line of shores in bay 7-8 extended completely across the 16th floor.

Eyewitnesses do not agree on the interior shoring conditions under the east side of the roof slab. While none of those who saw the initial sagging of the roof recalled seeing shores in the area of the sagging, some felt that there were occasional shores at other interior locations.

B. 15TH FLOOR TO 16TH FLOOR

Figure 7.15 shows the locations of twenty-three 4 x 4 inch wood shores which were present under the western uncollapsed portion of the 16th floor. The photographs in Figures 2 and 6 of Part I indicate that there were perimeter shores under the east side of the 16th floor, but no attempt was made to determine from eyewitnesses the interior east side shoring conditions at this level.

C. ROOF TO MECHANICAL ROOM FLOOR

Figure 7.16 is a photograph taken inside the crawl space between the roof and mechanical room floor on the uncollapsed west side. The formwork and shoring shown in this photograph is typical of that under all of the uncollapsed mechanical room floor. There is no evidence to indicate that the conditions on the east side were unlike those shown in Figure 7.16.

FIGURE 7.1 LOCATIONS OF SHORES: 16TH FLOOR TO ROOF ON THE UNCOLLAPSED WEST SIDE

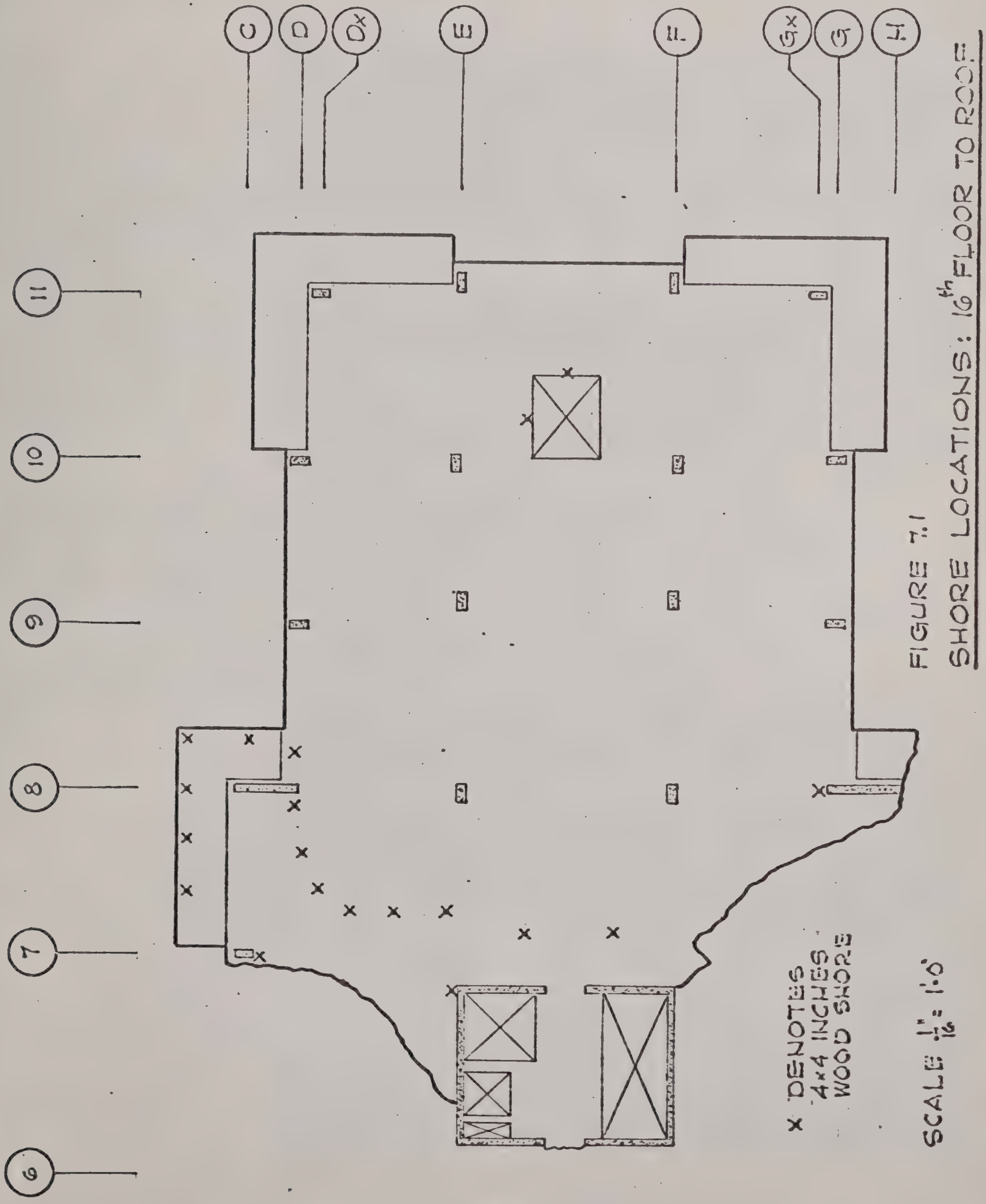

C
D
Dx
E
F
Gx
G
H
11
10
9
8
7
6
FIGURE 7.1
SHORE LOCATIONS: 16th FLOOR TO ROOF
JANUARY 25, 1971
x DENOTES 4x4 INCHES WOOD SHORE
SCALE 1/16" = 1'-0"

FIGURE 7.2 16TH FLOOR: STANDING AT F-11 FACING EAST SOUTHEAST

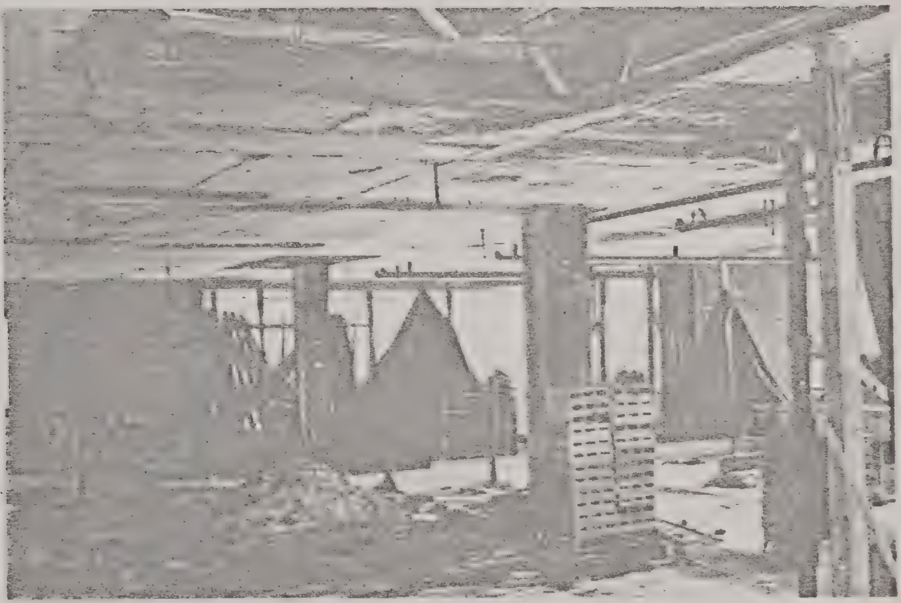


FIGURE 7.3 16TH FLOOR: STANDING AT E-11 FACING EAST NORTHEAST

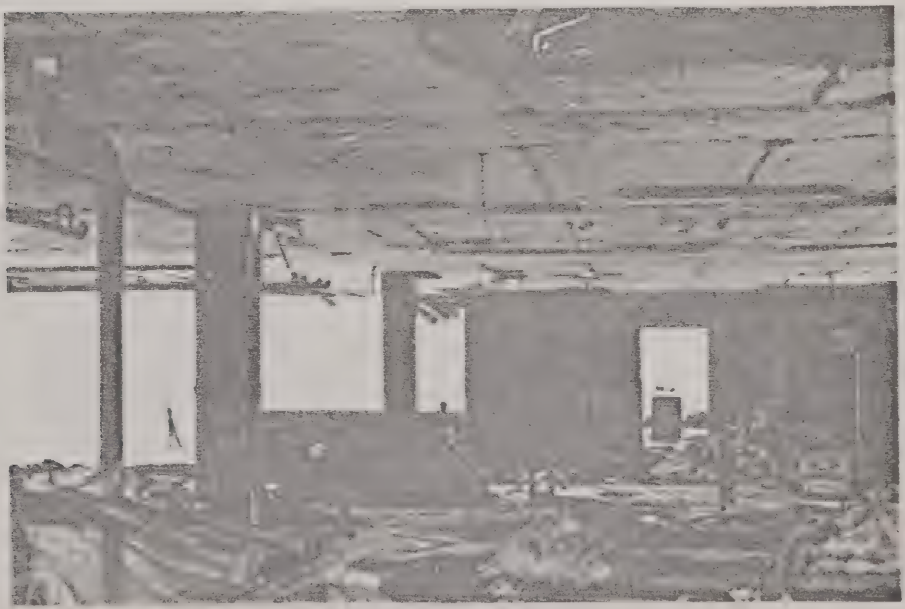


FIGURE 7.4 16TH FLOOR: STANDING AT E½-9 FACING SOUTHEAST

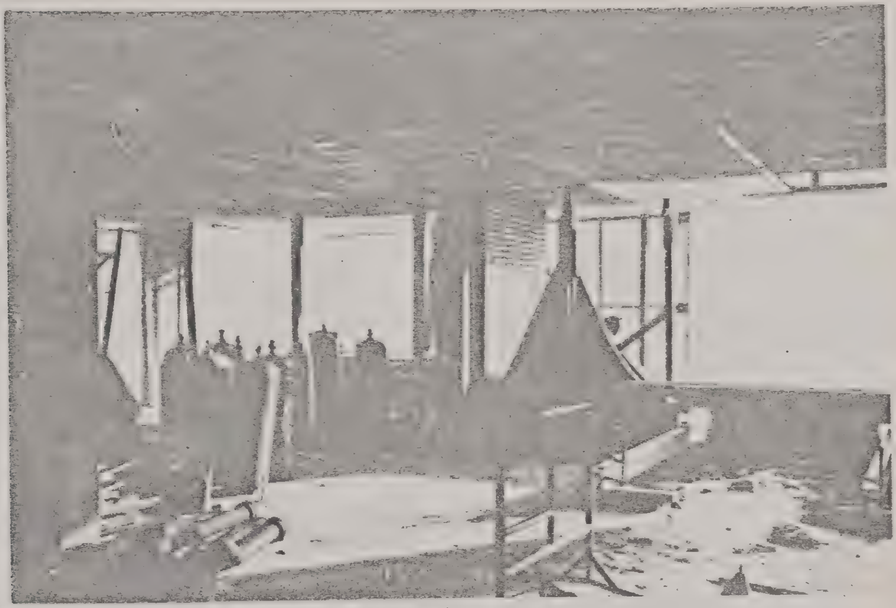


FIGURE 7.5 16TH FLOOR: STANDING AT F-8½ FACING SOUTHEAST

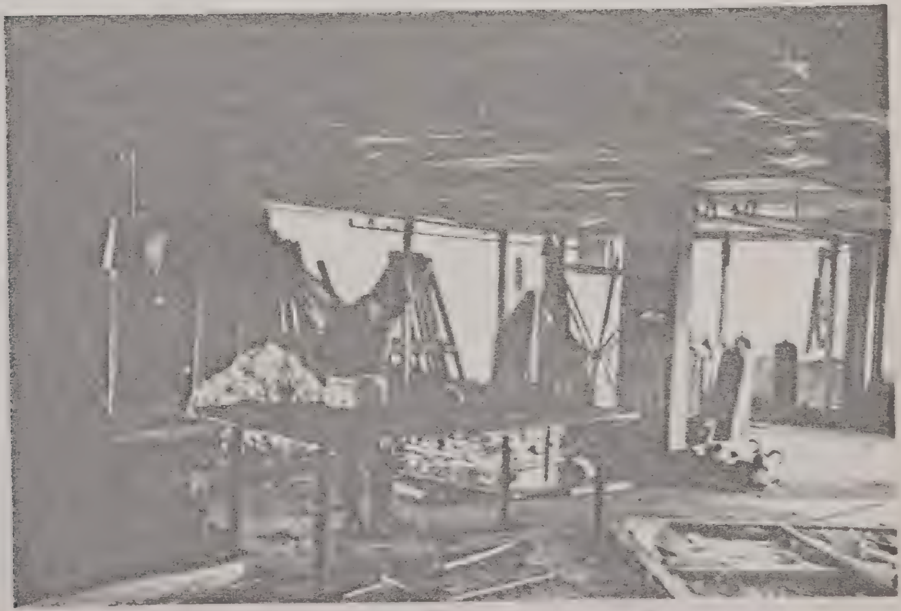


FIGURE 7.6 16TH FLOOR: STANDING AT F$\frac{1}{4}$-9 FACING EAST SOUTHEAST

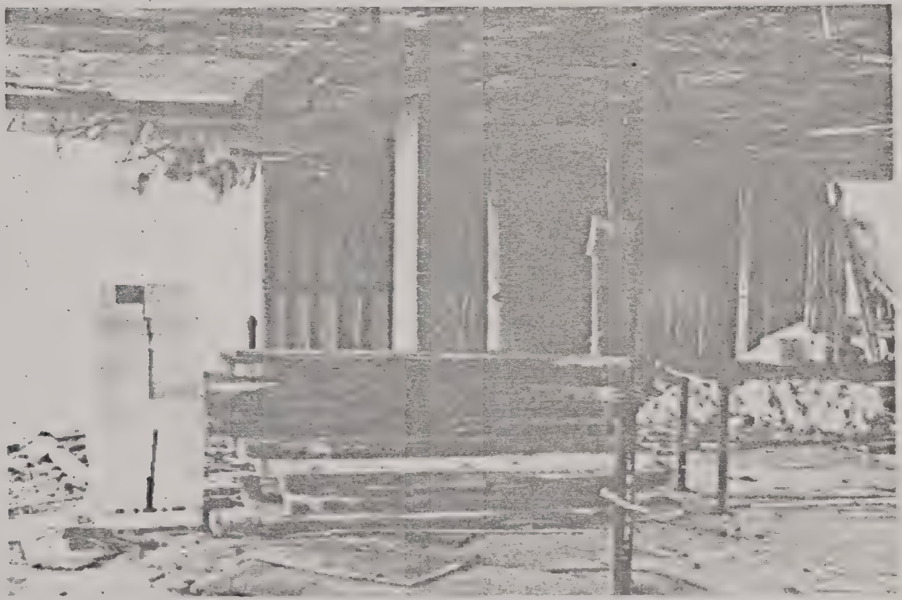


FIGURE 7.7 16TH FLOOR: STANDING AT F$\frac{1}{2}$-9 FACING EAST

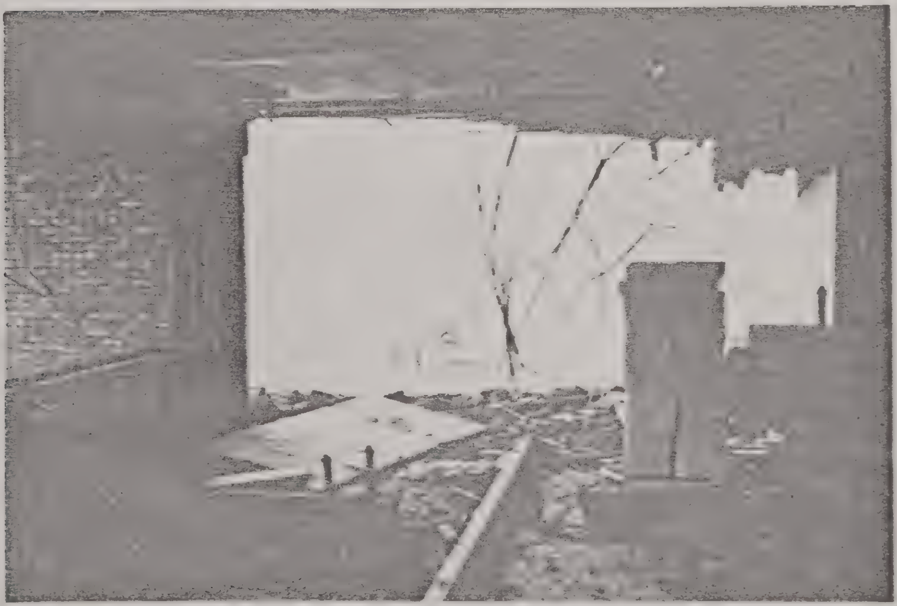


FIGURE 7.6 16TH FLOOR: STANDING AT F$\frac{1}{4}$-9 FACING EAST SOUTHEAST

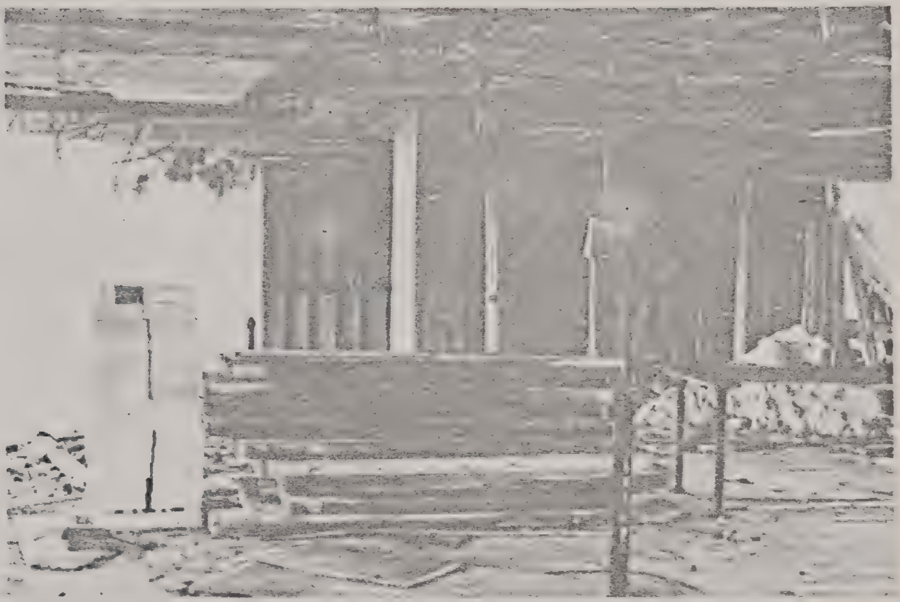


FIGURE 7.7 16TH FLOOR: STANDING AT F$\frac{1}{2}$-9 FACING EAST

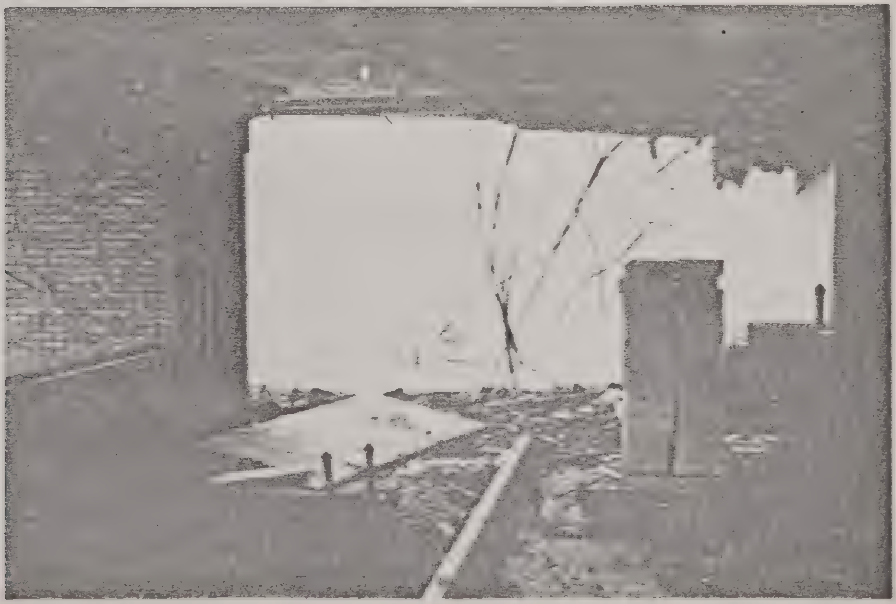


FIGURE 7.8 16TH FLOOR: STANDING AT F-7¾ FACING SOUTH SOUTHEAST

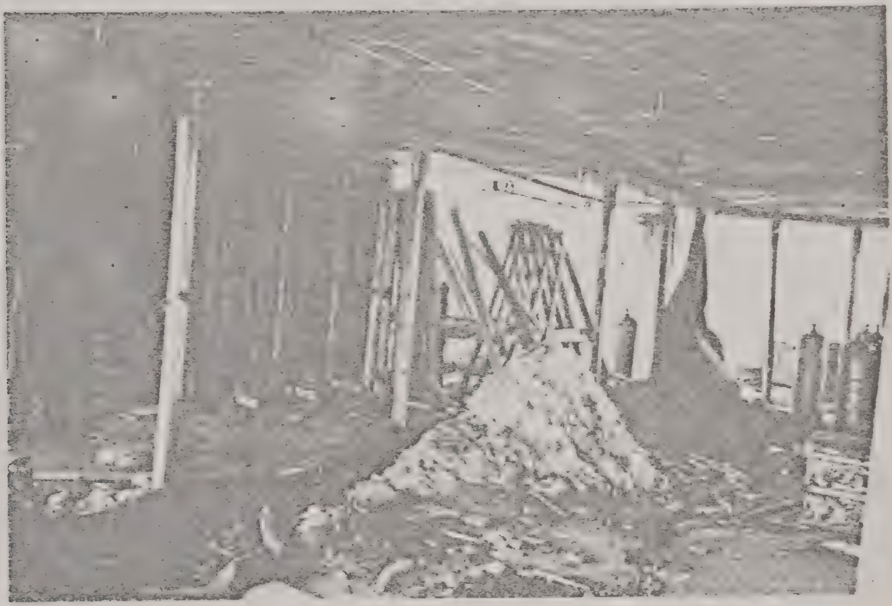


FIGURE 7.9 16TH FLOOR: STANDING AT E¾ - 7¾ FACING NORTHEAST

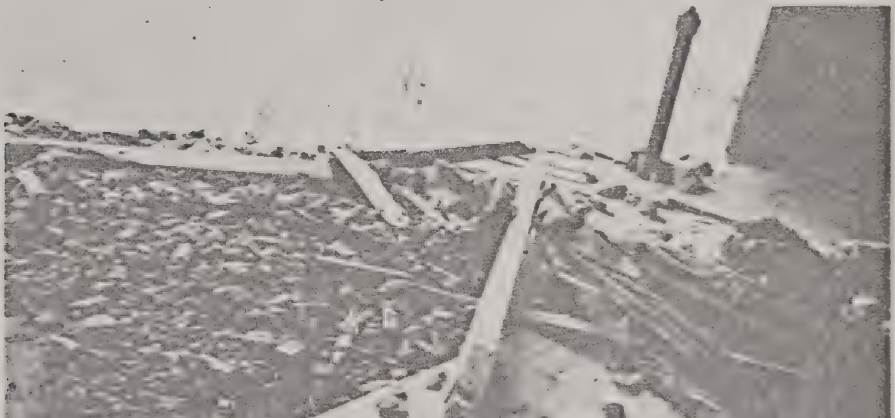


FIGURE 7.10 16TH FLOOR: STANDING AT $E\frac{3}{4}$-$7\frac{3}{4}$ FACING EAST

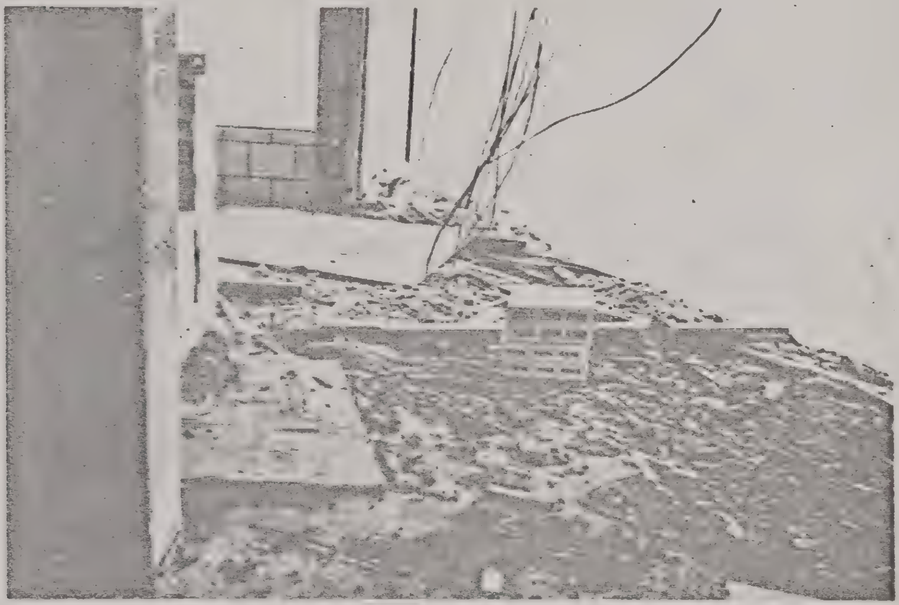


FIGURE 7.11 16TH FLOOR: STANDING AT $E\frac{1}{2}$-8 FACING WEST SOUTHWEST

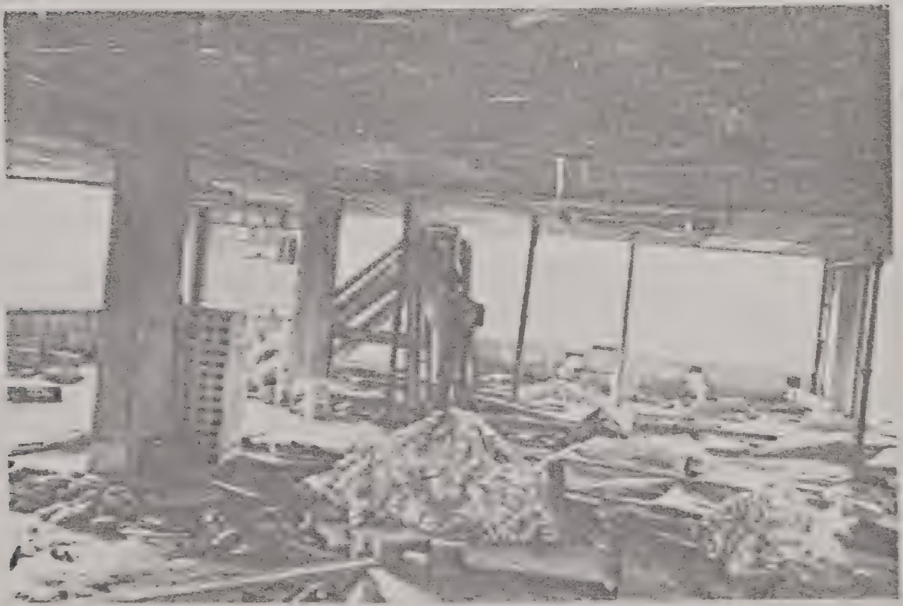


FIGURE 7.12 16TH FLOOR; STANDING AT E$\frac{1}{2}$-8 FACING WEST NORTHWEST

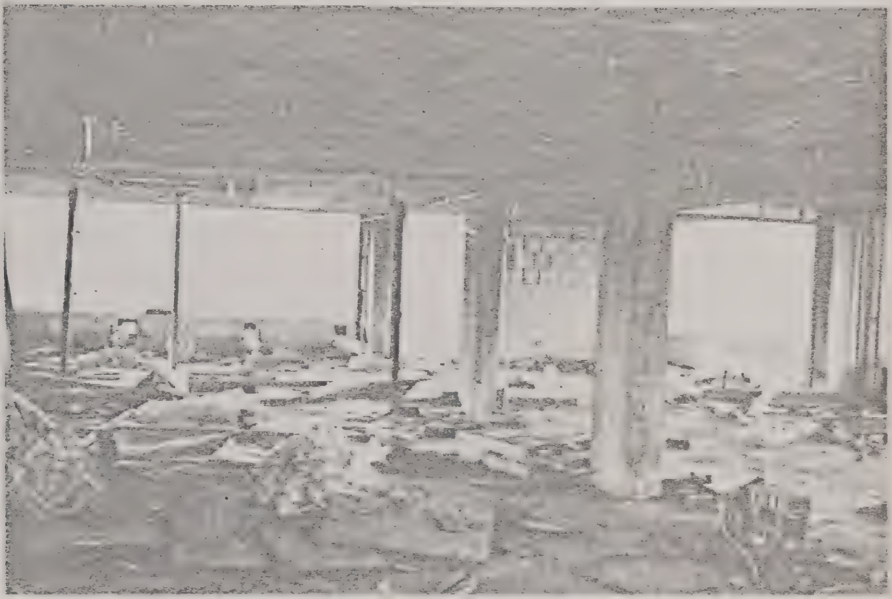


FIGURE 7.13 16TH FLOOR: STANDING AT ELEVATOR CORE OPENING FACING TOP OF SHORE AT E$\frac{1}{4}$-7$\frac{1}{2}$

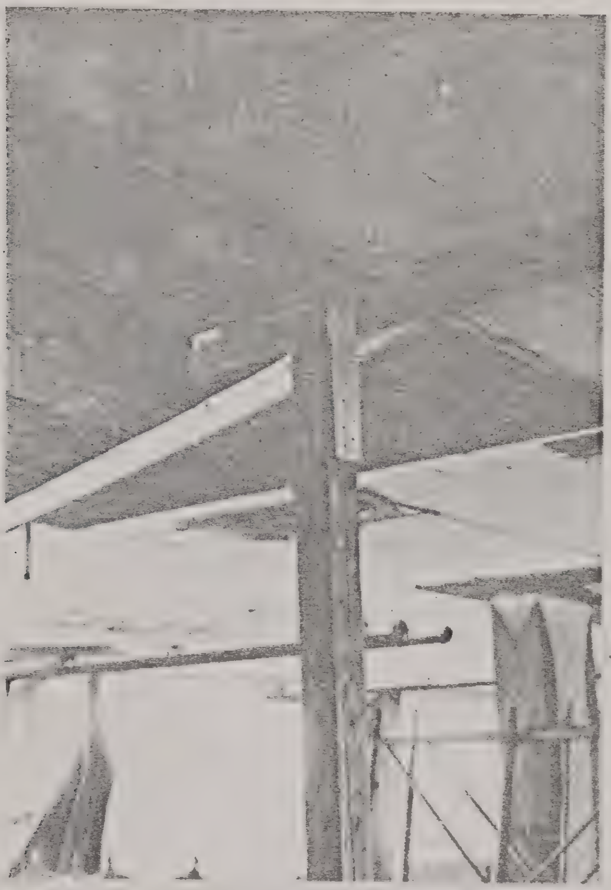


FIGURE 7.14 16TH FLOOR: STANDING AT ELEVATOR CORE OPENING FACING BOTTOM OF SHORE AT $E\frac{1}{4}$-$7\frac{1}{2}$

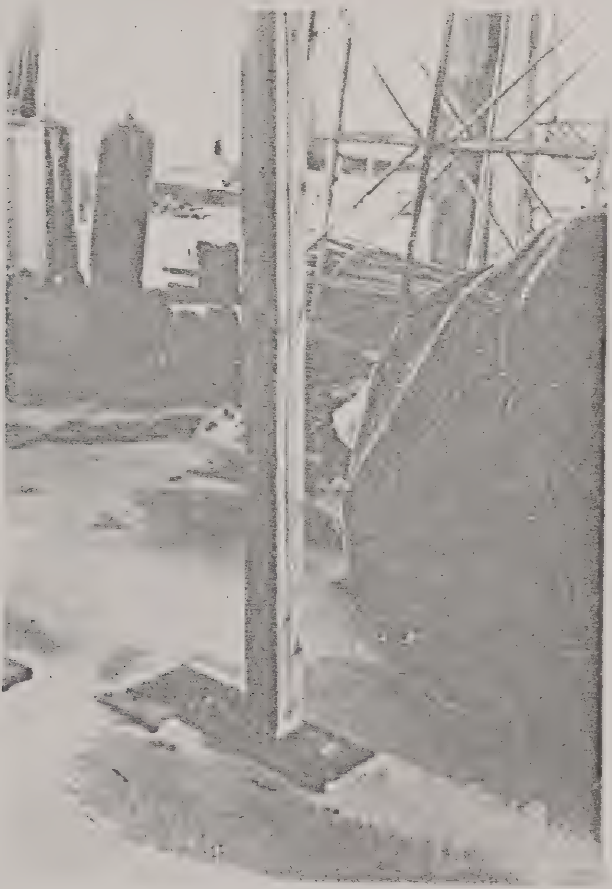


FIGURE 7.15 LOCATION OF SHORES: 15TH FLOOR TO 16TH FLOOR ON THE UNCOLLAPSED WEST SIDE

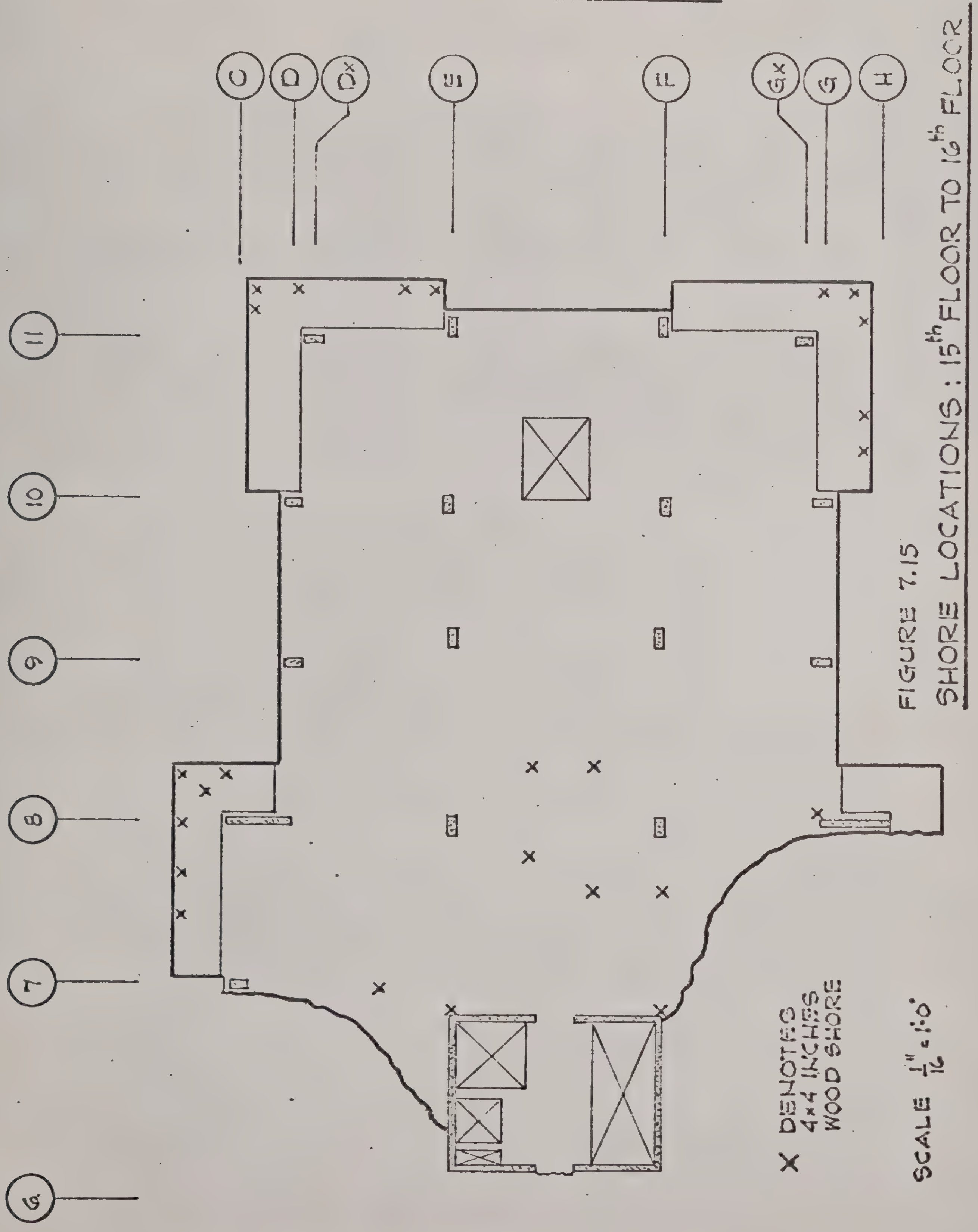
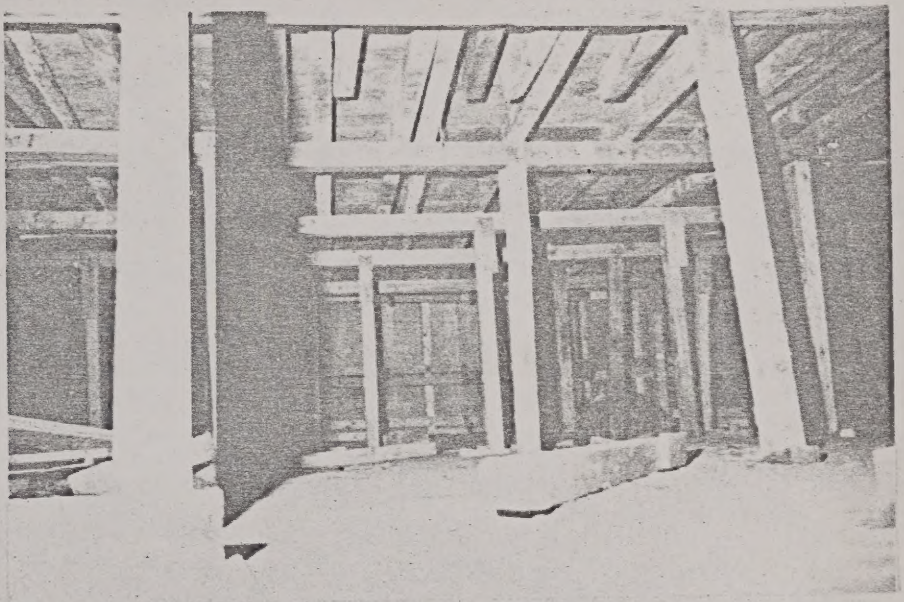

C
D
Dx
E
F
Gx
G
H
11
10
9
8
7
6
FIGURE 7.15
SHORE LOCATIONS: 15th FLOOR TO 16th FLOOR
JANUARY 25, 1971
X DENOTES 4×4 INCHES WOOD SHORE
SCALE 1/16" = 1'-0"

FIGURE 7.16 FORMWORK AND SHORING: ROOF TO MECHANICAL ROOM FLOOR ON THE UNCOLLAPSED WEST SIDE



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BF-RED
BG-BLACK
BD-LT. GREY
BP-LT. GREEN
BU-LT. BLUE
BY-YELLOW

BL-GOLD
BQ-PALM GREEN
BX-EXECUTIVE RED
BR-BROWN
BA-TANGERINE
BB-ROYAL BLUE

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